ENGINEERING NEWS

A JOURNAL OF CIVIL, MECHANICAL, MINING AND ELECTRICAL ENGINEERING

Vol. 66. No. 14. 220 Broadway, New York, October 5, 1911.

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The Columbia River Bridge of the North Coast Railroad Co. (now the Oregon-Washington R. R. & N. Co.)

By F. L. PITMAN,* M. Am. Soc. C. E.

A few years ago the North Coast Railroad Co. was organized to build a direct line across the State of Washington from Spokane to a point midway between Seattle, Wash., and Portland, Ore., with lines to both these cities. Two branches were to form direct lines also between Spokane and Walla Walla and between North Yakima and Walla Walla, all in Washington. In addition, a number of other branches were designed to reach the extensive territory of the "Inland Empire" and along the Columbia River. In 1910, the company was absorbed by the Harriman System and its lines became a part of what is now the Oregon-Washington Railroad & Navigation Co. The proposed western part of the new line (extending north to Seattle and south to Portland) was then abandoned, connection being made with the existing line, which extends down the south bank of the Columbia River to Portland, and thence north to Seattle. The line from North Yakima to Walla Walla was built, however, and is now in operation.

The original plans of the North Coast R. R. Co. included a crossing of the Columbia River. Accordingly, one of the first problems confronting the engineers of the company was the selection of a proper crossing of this river, within the limits of permissible deviation from the route adopted by the company for its railway. The division of the railway involving the crossing extended from North Yakima, Wash., to Walla Walla, and was projected to cross the Columbia River in the neighborhood of Kennewick, Wash. Accordingly, the river was carefully reconnoitered for a distance of 15 miles above and 20 miles below the point named.

Appreciating the importance and value of the site itself, a small party was organized for the purpose of detailed examination of the river over this distance. Several sites were carefully surveyed, the work including soundings, character of river bed, direction and velocity of current, course and alinement of the river banks, location of islands or bars, and other data bearing upon the value of the site. After the information obtained by this small party had been plotted and tabulated, the different sites were compared, and that one selected which presented the most favorable showing, considering all features. Much weight was given to the importance of a straight river approach, and to a uniform width and depth of stream. A straight approach was sought for a distance of at least one mile above the proposed structure, thus insuring a current free from variations in its direction at the different stages of water. Consideration was also given to the fall of the river in the locality of the proposed crossing, the object being to select a site which would be as free as possible from greatly increased current velocities during high water. Preference was given, also, to those sites which afforded liberal but not excessive depth of water for the entire distance across the water surface. It is, of course, understood that much importance was attached to the character of the river bed itself.

The examinations above outlined required a period of several months. The data, when compiled, showed results in favor of a crossing approximately two miles below the mouth of the Snake River, and six miles below the city of Kennewick. This site has many features in its favor, among them being the following:

(1) The river bed is composed of a bluish soapstone rock, which, for about one-third of the distance in the center of the stream, is swept clean and smooth by the current. Over the remaining distance, a layer of boulders, gravel and sand, in a cemented condition, is deposited upon the bed-rock formation; this increases in depth toward each side of the river, reaching a maximum of some 10 ft. upon the west side.

(2) The river is absolutely straight for a distance of two miles above, and more than one mile below, the selected crossing.

(3) The water surface distance, though not excessive, is fully the average width of the river, and the maximum depth is less, while the average depth is greater than the average of the stream.

(4) Practically a right-angle crossing was obtained, although curvature was required to secure this result.

(5) The current of the stream is checked by a solid rock ledge extending entirely across the river bed some two miles below this site. The effect is to spread the river to more than double its normal width; and as the elevation of the ledge is considerably above the uniform gradient of the river, the current is retarded in times of flood, thus appreciably decreasing its velocity for a considerable distance above this outcropping ledge.

(6) The river is entirely free from islands or other seriously disturbing elements influencing the direction of the current. There are, however, some bars or shoals above the bridge site, which cause the direction of the current to fluctuate slightly at different stages of the river.

All the foregoing items were given careful consideration in deciding upon the preferable crossing. Notwithstanding the fact that such details as are above named are frequently accorded minor attention, or altogether disre-

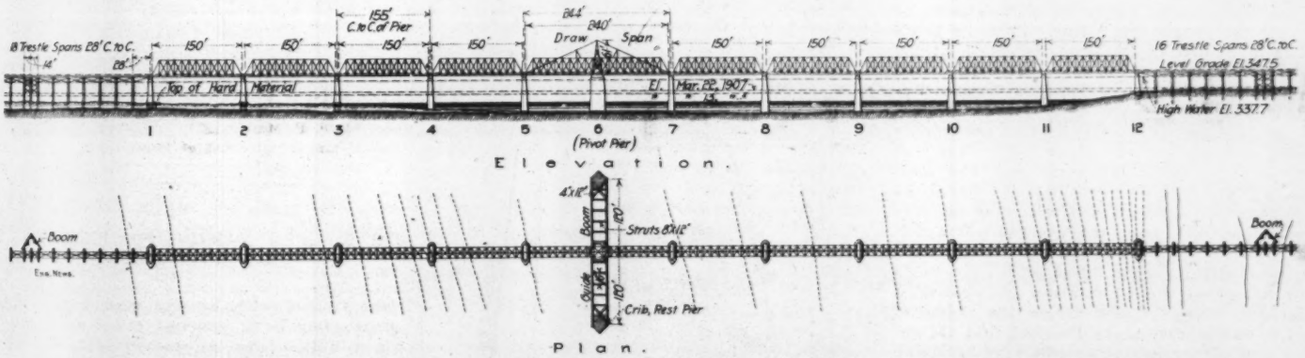

FIG. 1. ELEVATION AND PLAN OF THE COLUMBIA RIVER BRIDGE. A TIMBER BRIDGE ON CONCRETE PIERS.

*Chief Engineer, Oregon-Washington Railroad & Navigation Co. (Third District), Spokane, Wash.

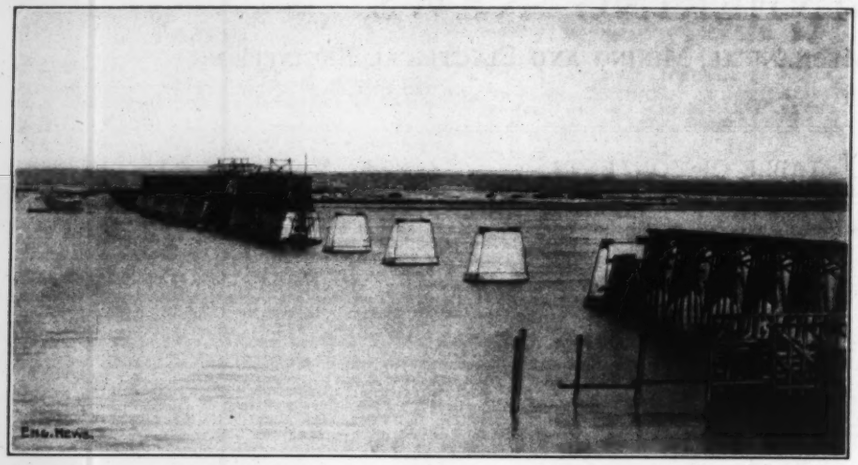

FIG. 2. CONSTRUCTION OF THE COLUMBIA RIVER BRIDGE; NORTH COAST R. R. (OCT. 13, 1910).

garded in the selection of bridge sites, it is the opinion of the writer that the fullest data of this character should be developed; and all of the features enumerated should be weighed carefully in connection with the selection of important river crossings.

Few persons who have not seen the Columbia River really appreciate its magnitude. While it is not the longest and possibly not the largest, it is beyond question the mightest river in the United States. Its width ranges from a few hundred feet to more than seven miles; and the maximum depth of any cross-section, from 10 or 20 ft. to several hundred feet. It is a swift-flowing stream of clear water, with beaches of heavy boulders, gravel and sand, which the rushing waters of the stream beat and pound into a more or less cemented condition sufficiently hard to withstand the impact of water and drift during flood, and the grinding of ice gorges which are sometimes formed by the rapid flowing current. The channel is pronounced, and seldom changes.

The Columbia River is unlike many of its sister rivers in that it possesses, throughout its course, little respect for spurs of mountains, or even for the ranges themselves. From its headwaters in British Columbia to its mouth at the westerly boundary of the states of Oregon and Washington, it has found passages through numerous mountain spurs and gigantic ridges, and in seeking its outlet to the sea it has pierced even the Cascade range itself. The waters of the Columbia are very clear and cold; even in times of flood very little silt is carried in suspension. Altogether, it is a most remarkable stream, and upon the average offers difficult problems for solution by the bridge engineer who seeks to span its current.

The extent of the undertaking of building a bridge for railway traffic across the Columbia is, in a way, expressed by the fact that the reputed cost of the two structures (one belonging to the Northern Pacific Ry. and the other to the Chicago, Milwaukee & Puget Sound Ry.) some miles above the bridge proposed by our company, reached an outlay stated in round figures at $1,000,000 each. This bridge was, therefore, a matter requiring careful consideration, both as to the character of structure which should be erected and the fitness of the site selected. As to the character of the structure, the four types receiving special attention were:

(1) A steel bridge, having a grade line elevation sufficient to allow clearance between the water line and lowest steel for the passage of river boats. This clearance was fixed by the War Department at 60 ft. above ordinary high water.

(2) A steel bridge with concrete piers; the bridge to have a grade line elevation sufficient only to safely avoid high water; traffic to be provided for by either a swing or rolling-lift draw span.

(3) A timber bridge, with concrete piers; the bridge to have a grade line elevation sufficient only for protection against high water; the river traffic to be taken care of by a timber swing span.

(4) A timber bridge with crib piers filled with loose rock; the bridge to have a grade line elevation sufficient only for protection against high water, and a timber swing-span to provide for river traffic.

In view of the fact that concrete piers would provide a permanent substructure, and because of the facility with which the temporary timber superstructure could at any future time be replaced with steel, it was decided to adopt the third treatment of those above named. Estimates of cost of piers fixed the length of spans slightly above 150 ft. Accordingly, a superstructure was adopted consisting of nine simple spans, each 150 ft. long, and one 240 ft. draw-bridge, all of the timber Howe truss type. The 240-ft. draw provides two clear openings of 100 ft. for the passage of boats. The eleven spans described aggregate 1,639 ft. in length. The remaining distance between abutments is covered by timber trestle approaches; the one upon the east is 447.5 ft., and that upon the west is 503.5 ft. in length. The spans and trestle approaches were designed to carry a live load of two 168-ton consolidation (2-8-0) locomotives followed by a uniform load of 5,000 lbs. per linear foot of bridge. Fig. 1 shows the elevation and plan of the bridge as built, and the general dimensions are set out upon this drawing. Fig. 2 is a view looking across the river and showing the construction work in progress.

Careful foundation tests were made to determine the treatment which would be followed in placing the piers upon the river bed. It was impossible to penetrate the soapstone formation with a pile provided with a sharp-pointed steel shoe, or with a railway rail driven by careful blows from a 3,000-lb. hammer. Over those portions of the river bed covered by the deposit of cemented boulders, gravel and sand, piling could be "coaxed" into the material by a great number of light blows. A test pile, having a diameter of 13 ins. at the lower end, which received more than 600 blows, penetrated to a depth of 4½ ft. A pull at the top of this pile caused it to break off at the surface of the river bed, rather than to heel out the compact material into which it was driven.

Notwithstanding this test, four of the piers

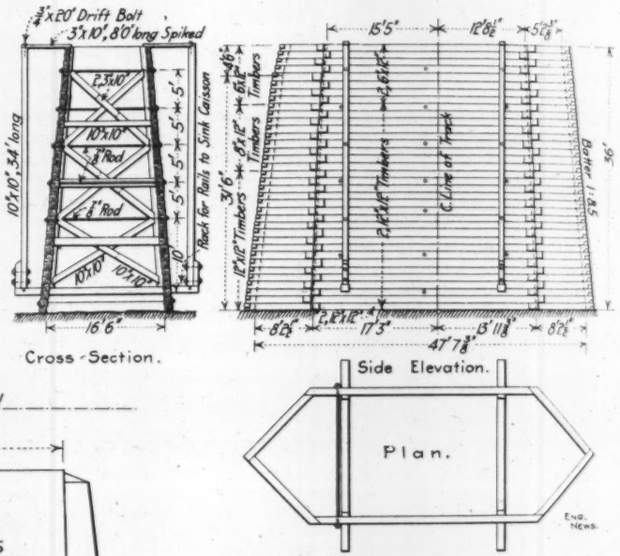

FIG. 3. OPEN CAISSONS FOR PIERS; COLUMBIA RIVER BRIDGE.

were treated with bearing piles driven at approximately 3 ft. centers; these were cut off about 2 ft. above the ground surface. It was considered that these piles would serve a purpose as dowels if they did not add greatly to the bearing capacity of the piers. By the time piles had been driven for four piers, however, careful examination led to the conclusion that a disturbance of the cemented condition of the boulders, gravel and sand, caused by the driving of piles, decreased (rather than increased) the bearing capacity of the river bed in its natural condition. Accordingly the use of bearing piles was thereafter dispensed with.

The maximum depth of water at the ordinary stage is about 30 ft., and the maximum velocity of the current is about seven miles per hour. The construction of concrete piers under these conditions presented a problem of considerable magnitude. The result was accomplished, however, by the use of open caissons. These consisted of 12 × 12-in. timbers, framed and braced to conform with shape of piers as shown in Fig. 3. After being framed, each caisson was floated into position, and sunk by loading with steel rails. The rack for the rails shows the method

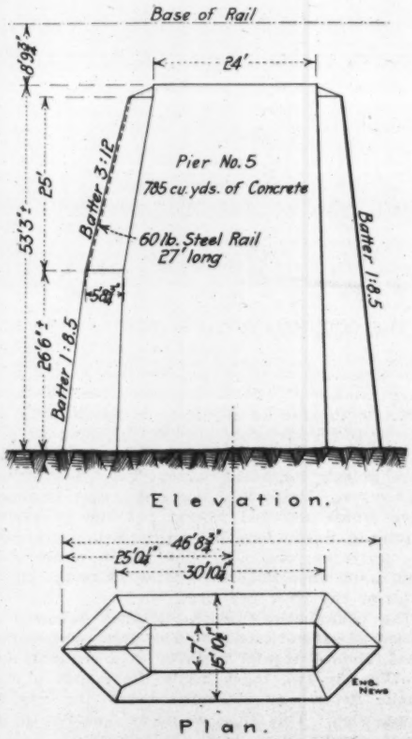

Fig. 4. Typical Pier for the Fixed Spans; Columbia River Bridge.

of keeping the load near the bottom, to avoid capsizing. After the caisson had settled to the proper position, and an additional weight of rails placed on the top, to insure stability, the depositing of concrete was begun in the still water within the caisson, without any attempt at pumping.

The depth of water through which concrete was thus placed reached a maximum of 32 ft. Large buckets, with movable bottoms, were used for the purpose, thus permitting the deposit of a reasonably thick layer, which was carried continuously over the area of the pier. Careful examinations were made as the work progressed, showing that only a very slight proportion of the cement was being dissolved and carried away by the water; but to offset the loss, the proportions were increased from 1:2½:5 to 1:2:4. Variations in the stage of the water permitted a careful examination of concrete placed in this manner; and these, combined with tests made under water, proved the satisfactory quality of the concrete contained in the piers below the water surface. After completion of each pier, all 12 × 12-in. timbers extending appreciably above low water were removed, together with the forms for the upper portion of the pier. Fig. 4 shows the design of the piers.

The timber spans are of ordinary Howe truss design, and the drawings of the trusses for the fixed and draw spans are shown in Figs. 5 and 6. The turntable of the draw span, with operating machinery and end lifts are shown in Fig. 7. The trusses rest on blocking upon the piers as shown in Fig. 8. It will be seen that there is no coping course on the piers, but this would be built in the event of the future renewal of the timber spans with steel spans.

All timbers used in the construction of the bridge were of Washington fir, and the life of the timber approaches and truss spans is estimated at 8 to 12 years. All joints and framed portions of the trusses were treated with a liberal coating of red mineral paint mixed with linseed oil, and the entire timber portions were given one thorough coat of this material. The estimates show that the cost of steel superstructure, as compared with timber, would have been 50% more.

The great depth of water, together with the character of river bottom, made it expensive to build long protection piers, above and below the pivot pier. Instead of these, two small piers were used, shown on Fig. 1 as crib rests. They were constructed of 12 × 12-in. timbers, placed in the same manner as the pier caissons, and filled with loose rock. The upstream ends are well covered with sheet iron near the low water line, for protection against drifting ice. A floating timber boom, of the design shown in Fig. 9, is placed for the protection of boats against the effect of cross-currents and wind in passing through the draw.

The extension of approaches into the river channel made necessary the use of an unusually heavy trestle structure. It was deemed advisable to make the spans as long as practicable, and these were fixed at 28 ft. Each bent of the trestle contains seven piles, sway-braced down to high water line in the usual manner. From high water to low water both sides are completely covered with 3-in. planking, placed diagonally, and securely spiked to each pile. This is shown in Fig. 10. Caps of extra length were provided to act as falsework at some future time, should it be deemed advisable to replace the trestle with short girder or other spans. Under each track rail there are three 14 × 28-in. stringers, held together with packing bolts, and also bolted to the caps. In addition to the bracing shown in elevation, each panel contains also a set of lateral braces.

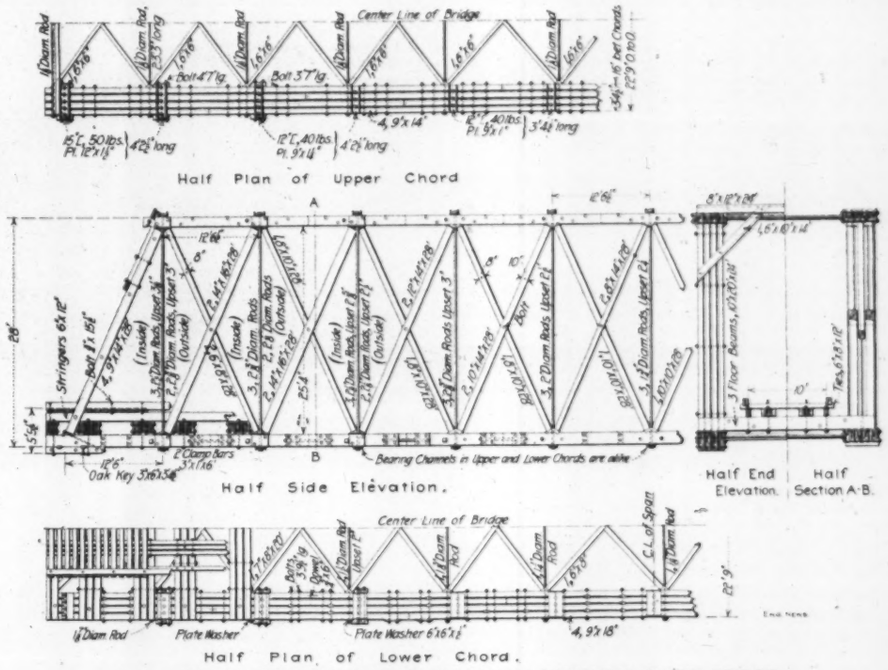
Half Plan of Upper Chord
Half Side Elevation.
Half End Elevation.
Half Section A-B.
Half Plan of Lower Chords.
Bearing Channels in Upper and Lower Chords are alike.

FIG. 5. 150-FT. TIMBER HOWE-TRUSS SPAN; COLUMBIA RIVER BRIDGE.

During construction of the approaches and piers, the materials for the spans were assembled, and the timber framed in a yard conveniently near the bridge site. Floating trussed falsework, designed by the contractors, was in readiness, and the erection of trusses required a comparatively short time. A rather remarkable erection record was obtained in the placing and completion of one of the 150-ft. spans in 12½ hours of time. The views in Figs. 11 and 12 give an idea of the design of falsework, and some of the methods of handling materials during erection. The deck-truss falsework is supported on two scows, which also carry the hoisting engines. Above its upper deck is a construction track on a low-trestle, and the upper part of the structure forms the erecting framework. In Fig. 11 the falsework is in position for one of the spans adjacent to the drawspan, and the men are standing upon one of the built-up timber chords for this 150-ft. span. In Fig. 12 the falsework is being moved to the span at the other end of the draw.

The Columbia River, similarly to the Nile, is subject to an annual high-water period, beginning in June and subsiding in August. This occurs with almost rigid regularity, both as to time and volume. The construction of piers extended over two of these periods. A part of them were exposed to the flood waters of two seasons, and all of them to one high water period, before the placing of rip-rap. The river bed upon which the piers rest showed a degree of resistance such that no appreciable erosion was detected as a result of the flood waters of the two seasons referred to. Since then, however, there has been deposited a liberal treatment of heavy angular rock rip-rap around each of the piers, the same extending to a height of at least 10 ft. above the original surface at the river bed, and sloping from the pier in all directions, at about 1 on 2 or 2½. It is believed that this rip-rap, together with the proved hardness of the river bottom, insures the stability of the piers indefinitely.

The quantities contained in the bridge are as follows: Piling, 2,000 lin. ft.; timber, 1,812,000

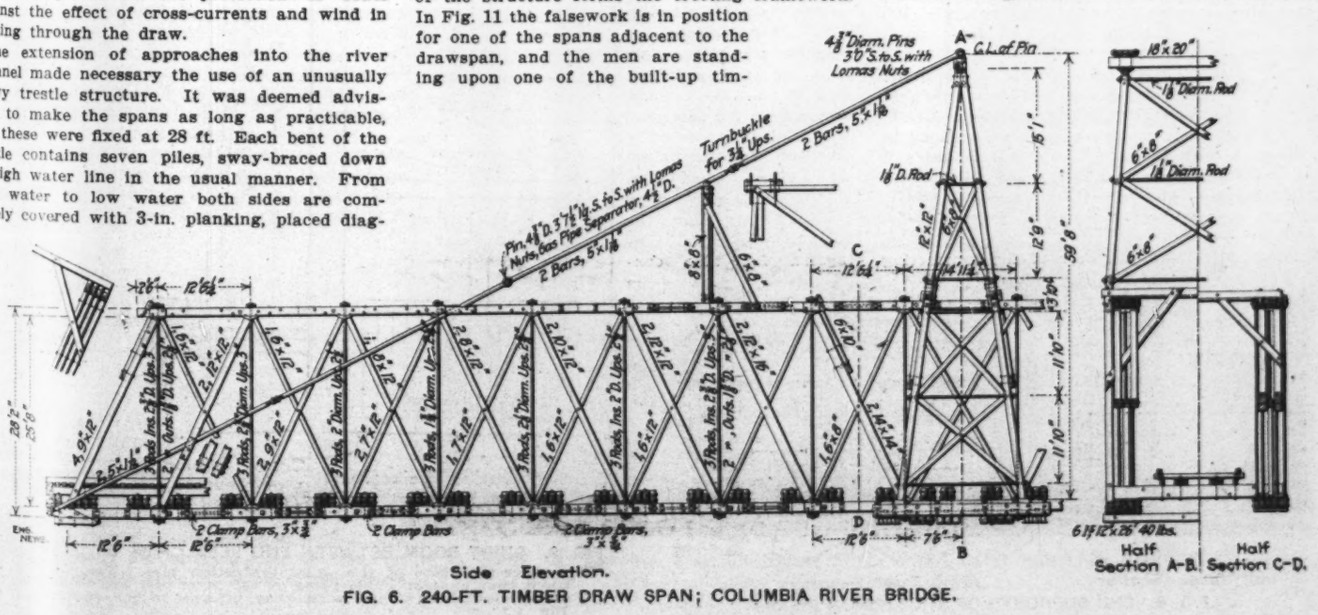

FIG. 6. 240-FT. TIMBER DRAW SPAN; COLUMBIA RIVER BRIDGE.

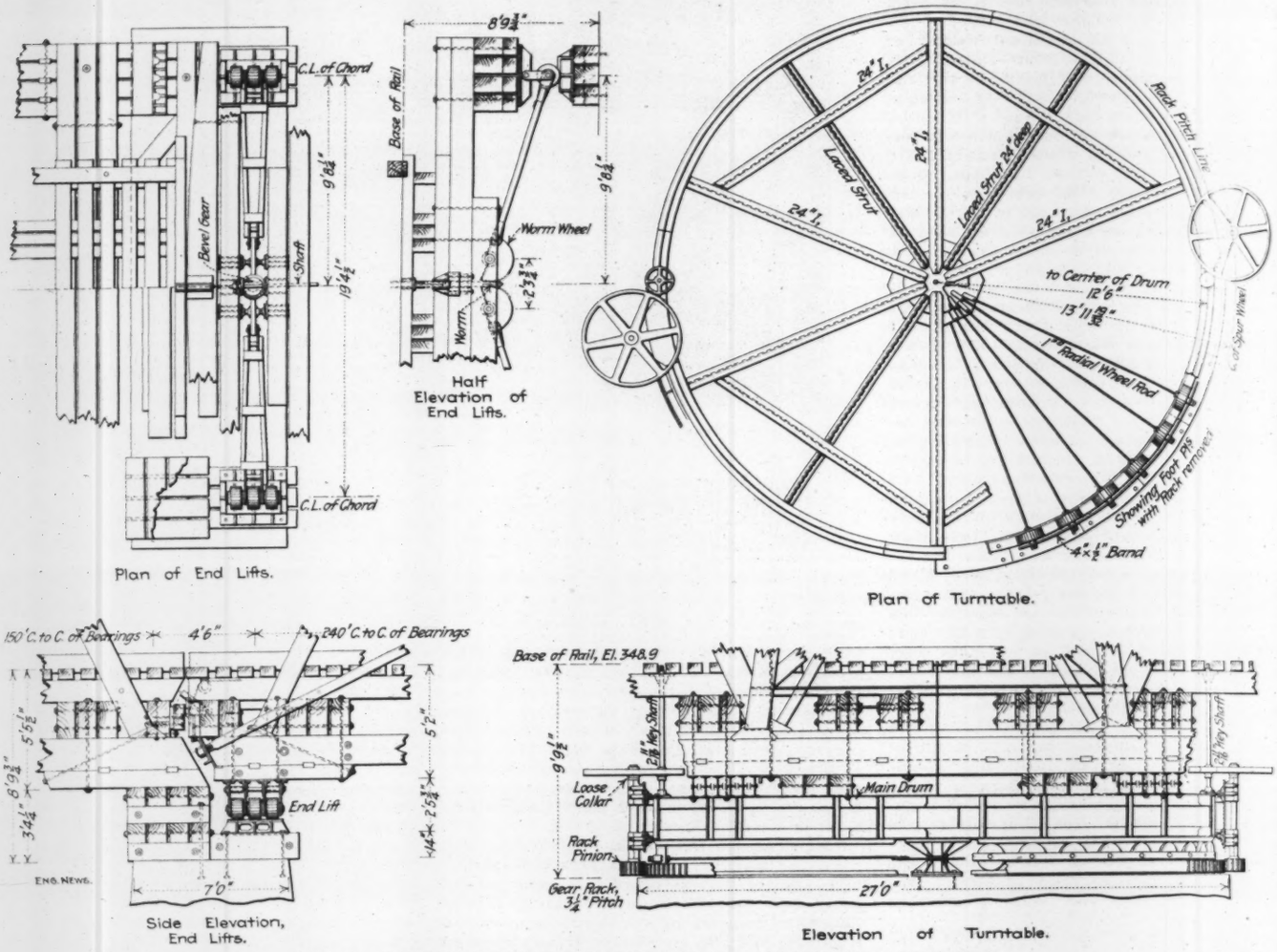

FIG. 7. OPERATING MECHANISM AND END LIFTS OF 240-FT. DRAW SPAN.

ft. B. M.; iron, 1,215,400 lbs.; concrete, 10,000 cu. yds. From the date of beginning work until the opening of the bridge for traffic (Feb. 20, 1909, to Nov. 4, 1910), including, of course, delays occasioned by high water and other causes, appears quite a long period. But, if the actual working time is considered, the showing is much better. This working time was as follows: Installation of plant, 40 days; construction of piers, sinking caissons and placing concrete, 108 days; erection of bridge spans, 34 days; total, 182 days.

The contract for the construction of the bridge was let to the Puget Sound Bridge & Dredging Co., of Seattle, Wash., whose President, Mr. S. H. Hedges, made many valuable suggestions in connection with the plans for the structure and the execution thereof upon the ground. Mr. J. P. Livingston, who had charge as superintendent for the contractors, early demonstrated his superior qualifications as a practical bridge builder. Mr. Dean, engineer of the contracting company, and Mr. Dyer, vice president, also rendered valuable assistance in connection with the undertaking. For the railway company, Mr. A. D. McVay, Bridge Engineer, should be accredited with the detailed designs, and Mr. W. C. Marion, Assistant Engineer, and Mr. Wm. Collins, Superintendent of Bridge Construction, are each worthy of mention for the valuable services which they rendered in connection with the construction of this bridge. Especial mention should be made of the fact that Mr. Collins personally executed the construction of the approaches, with company forces. He, acting for the company, and Mr. Livingston for the contractor, literally carried out the plans upon the ground; and to their ingenuity is due the fact that the structure was completed, from beginning to end, without serious loss, mishap or failure of any kind.

The peculiar conditions of foundation and other

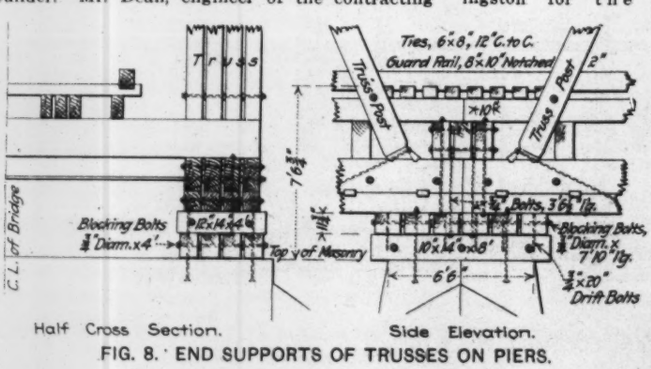

FIG. 8. END SUPPORTS OF TRUSSES ON PIERS.

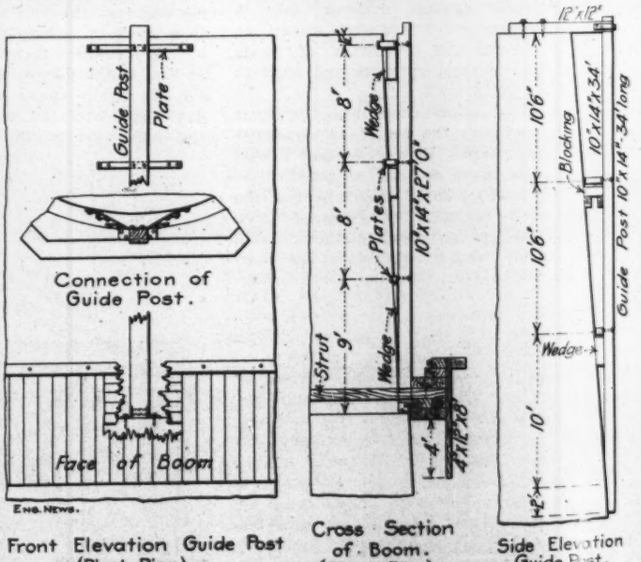

FIG. 9. GUIDE BOOM BETWEEN END REST CRIBS OF DRAW SPAN.
(This takes the place of the usual protection pier. The sides of the boom bear against guide posts on the pivot pier and crib rests, and are connected by struts, with sway bracing in the end panels; see Fig. 1.)

natural features of the bridge site have made possible the construction of this bridge, with results which should be especially emphasized. These results are as follows:

(1) Expenditures to the present time indicate that the structure can be extended and revised to include steel superstructure and concrete substructure between abutments, thus eliminating the present approaches and timber spans, within a total outlay of $550,000.

(2) The building of the bridge proceeded from beginning to end without loss of materials or expensive changes of plans, frequently encountered at less favorable sites.

(3) The substructure was successfully placed without the excavation of a single yard of foundation material or the pumping of a single gallon of water.

(4) The foundations have proved satisfactory as against high water without added and unexpected protection, so often an expensive item in bridge construction.

The foregoing advantages have not been made available without the exercise of careful judgment, which should be accredited largely to the employees above named, of both the railway company and the contractor.

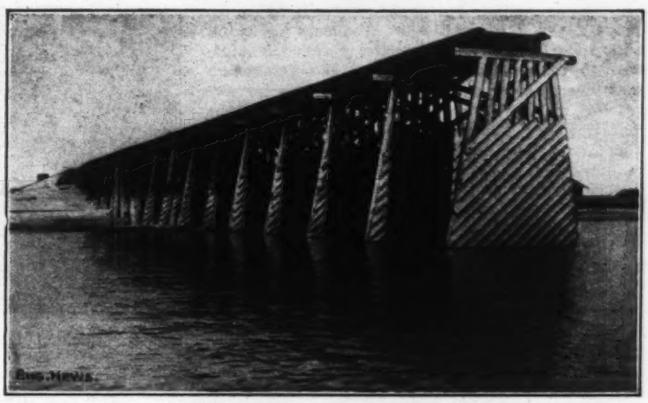

FIG. 10. EAST APPROACH OF COLUMBIA RIVER BRIDGE; (ALL BENTS SHEETED WITH DIAGONAL PLANKING).

The Effect of Pigments on Linseed Oil.

Mr. H. A. Gardner, of the Institute of Industrial Research, Washington, D. C., in the September issue of the "Journal of Industrial and Engineering Chemistry" of the American Chemical Society, reports a series of studies into the chemical activity of pigments ground in linseed oil. Mr. A. H. Sabin, Consulting Chemist, National Lead Co., has described (Journal of Ind. and Eng. Chem., vol. 3, p. 84) the peculiar action of various pigments in accelerating the drying of linseed oil. He accounted for the action as due to the hindering of peroxidation processes. Mr. Gardner cites his experiments as showing that several pigments in question, such as barytes and china clay, are really inert and have no chemical action, the stimulation of oxidation being accounted for by catalytic or contact effects.

Five-gram samples of various common pigments, including inert crystallized and amorphous types, were each ground separately in an agate mortar with five grams of raw linseed oil. Each of these pastes was placed in a marked beaker and allowed to stand for one month in a dustless atmosphere. The paste from each beaker was then extracted with benzine to remove the linseed oil. The oil solutions were heated to remove the benzine and the residue was burned to ash. The ash from each paste was weighed and if it ran above the percentage of ash determined on a blank sample of linseed oil (0.003%), the ash was analyzed qualitatively for metals. The following table shows the results:

Pigment.	Per cent. of ash in extract.	Analysis of ash.
Raw linseed oil without pigment	0.003	
Barytes	0.003	
Blanc fixe	0.003	
Silica	0.003	
Asbestine	0.005	
China clay	0.007	
Whiting	0.008	
Chrome yellow	0.025	Lead oxide (PbO)
Lithopone	0.031	Zinc oxide (ZnO)
Prussian blue	0.032	Iron oxide (Fe_2O_3)
Sublimed white lead	0.033	Lead oxide (PbO)
Zinc oxide	0.105	Zinc oxide (ZnO)
Corroded white lead	0.116	Lead oxide (PbO)
Red lead	0.2112	Lead oxide (PbO)

The slight increases with asbestine and china clay are explained by the carrying over of traces of pigment mechanically into the oil extract, these pigments being difficult to separate completely from the oil. Slight chemical action was apparent in the case of whiting and greater action in the still more alkaline pigments. The raw linseed oil used in the tests had a very low acid value, but the neutralization by alkaline pigments is held to account for a portion of the increased percentage of ash. It seems evident from these figures that the lead and zinc pigments are chemically active. The resulting saponification of the vehicle is apt to result in early disintegration. Mr. Gardner recommends the use of a moderate percentage of inert pigments in paints made of chemically active varieties, so that marked saponification would not take place.

Tests of Riveted Joints Under Impact are reported by E. Preuss in "Der Eisenbau" of September 1911. Two-rivet connections of three small rectangular plates were tested. The plates were superimposed, the middle one being offset upward so that it could be struck by a falling ram while the outer plates rested on an anvil block. The slip of the middle plate with respect to the others was measured, for blows of different heights and for various repetitions of the blow. Static pressure tests were also made and slip measured, for comparison with the impact tests. The conclusions reached by the author are:

The slip produced by a blow of given striking energy is only slightly increased by repeating the blow.

The slip is, roughly, a linear function of the energy of impact. In contrast herewith, in static tests the slip is a higher function of the loading.

For the same impact-energy per square inch of rivet-shear section, the slip was greater for larger (1-in.) rivets than for small (11/16-in. and ⅞-in.) rivets.

To produce a given amount of slip, the required static load in pounds is 8,000 to 22,000 times as great as the required impact energy in foot-pounds. This figure is greater for large rivets than for small, and is smaller for a great amount of slip than for a small amount of slip.

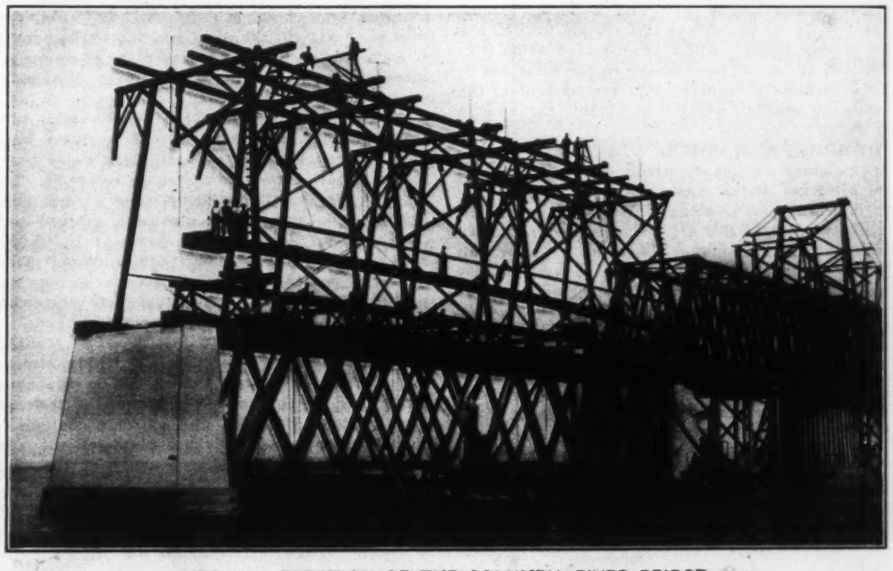

FIG. 11. ERECTION OF THE COLUMBIA RIVER BRIDGE.
(The erecting framework is carried by a pair of timber deck trusses supported on two scows.)

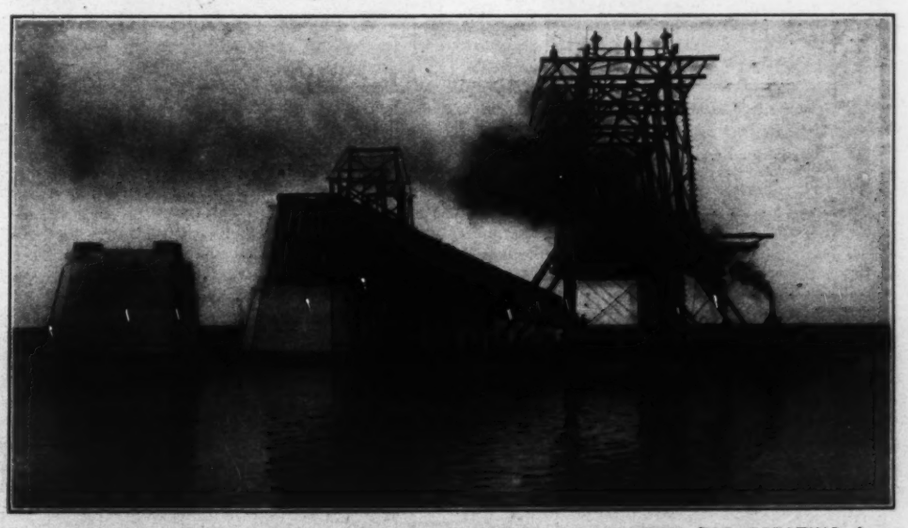

FIG. 12. MOVING THE ERECTION FALSEWORK INTO POSITION FOR ERECTING A NEW SPAN.

Water Purification Works at Toledo, Ohio.

By D. H. GOODWILLIE.*

Until the completion of its water purification works in February, 1910, the city of Toledo, Ohio, derived its water-supply directly from the Maumee River. Agitation for a more suitable supply resulted in the appointment of an engineering commission consisting of Messrs. Hazen, Benzenberg and Clark, whose recommendations led ultimately to the construction of the present mechanical filtration plant.

Included in the water purification works are a low-service pumping station, the filter plant proper—head house, filters and two coagulating and subsidence basins—and a clear-water storage basin, all located on the Maumee River, at a point 1½ miles south of the city limits (Fig. 1). Through a submerged wooden crib, a 48-in. cast-iron bell and spigot pipe and a 50-in. tunnel, water flows by gravity to the suction well in the low-service pump station, from which point it is lifted 65 ft. to the head house in the filter building. Through the purification system and to the high-duty station, 3½ miles distant, the flow is by gravity.

PUMPING AND POWER STATION.—The low-service pumping plant, which delivers water to the filtration works, has a total capacity of 35,000,000 gals. and is being increased to a nominal capacity of 50,000,000 gals. The pumps are placed in a pit 79 ft. in diameter, 48 ft. below the ground level. All the pumps are centrifugal. Besides one which has a capacity of 5,000,000 gals. and is motor driven, there are two two-stage centrifugal pumps of 15,000,000 gals. capacity, each direct connected to two 300-HP. Rathbun-Jones gas engines. The plant also includes two 150-HP. three-cylinder, four-cycle gas engines of the same manufacture, each direct-connected to 100-KW. General Electric generators, and two bituminous suction-gas producers built by R. D. Wood & Co. Two additional producers are being built to drive the new pumping plant. In addition to the small motor pump, the generators mentioned above supply current for the tar extractor motors, for lighting purposes, for the agitating and mixing motors in the filter house and for washing the filters. The wash load alone requires 300 amps. of 220 volts.

*Superintendent of Water Filtration, Toledo, O.

FILTERS AND BASINS.—The filter plant includes 20 units of 1,000,000 gals. daily rated capacity each, to which are being added 14 units (of equal capacity), all of the Norwood air-and-water-wash type. The filtering medium is a 30-in. layer of 4 to 10-mm. Haddonfield, N. J., sand placed above 9 ins. of graded gravel of four sizes. Hydraulic valves controlled from individual pedestals, instead of the usual operating tables, and an unusually commodious and accessible pipe gallery, are distinctive features of the filter plant (Figs. 2 and 3). Necessary chemical mixing and storage rooms, a machinery room for the wash apparatus, and two complete laboratories are also provided.

Untreated water is received in the head house and immediately coagulated and settled—no preliminary plain sedimentation being employed. The settling basins, each of 5,000,000 gals. capacity, are 500 ft. long and 100 ft. wide. The floors are made up of inverted groined arches and each basin is covered with a 5-in. reinforced-concrete roof of a modified Turner mushroom type. Above the roof is an 18-in. earth fill. For draining and cleaning purposes, two 30-in. box sewers of a 1 in 200 slope run the entire length of each basin. Under average rates of pumpage, a sedimentation period of from 6 to 8 hours is available in each basin.

Filtered water is delivered to the high-duty pump station by gravity through a 72-in. concrete conduit. A 5,000,000-gal. clear-water basin 250 ft. square serves as a storage reservoir. The floor of this basin is made up of inverted groined arches while the roof is also of a groined-arch construction. An additional basin of 16,000,000-gal. capacity is now under construction. This, with the present basin, will give a storage capacity of 21,000,000 gals., which is somewhat greater than the average daily consumption.

TREATMENT AND RESULTS.—The Maumee River is popularly known as a muddy stream. It is not the amount of the matter in suspension but rather its character which renders its treatment difficult. Sudden and extreme changes in turbidity, the finely divided state of the suspended matter and the persistency of the color more directly control the treatment than do turbidity values alone.

Fig. 3. Looking Down the Water Filter Room, Toledo, Ohio.

The average turbidity for the first full year of operation was 161 parts per million in the untreated water, all of which was completely removed in the purified effluent. The coefficient of fineness of the suspended matter averaged but .32 and in this factor lies the greatest obstacle to successful treatment. Maximum and minimum turbidity values of 2,500 and 6 parts per million, respectively, were noted, while the coefficient of fineness had but .5 as a maximum value. Color is not excessive. It ranges from 20 to 77 parts per million, with an average of 31 parts per million, but it is persistent and not readily susceptible to treatment.

Sufficient alkalinity to decompose all of the required alum is always present, averaging 161 parts per million, with a maximum value of 225 and a minimum of 56.

An unusual feature is the presence of monocarbonate alkalinity in large amounts. Values as high as 42 parts per million have been noted. In a water carrying a monocarbonate alkalinity and devoid of free carbonic acid it would seem that the use of iron and lime would be especially suitable, particularly from the standpoint of economy in lime consumption.

In the operation of the plant, the use of iron and lime was contemplated and provision for its use made. During the first three months of operation, both iron and lime and sulphate of alumina were given thorough trials. As a result, the use of iron and lime has been discontinued as being unsuitable to a water of this character. The addition of either lime or soda alone or together with iron or alum renders the removal of color more difficult. The larger the amounts of caustic, the more permanent the color "set." Alumina alone absorbs the color readily but its action both as to coagulation and to color removal is negligible until after the neutralization of the monocarbonate alkalinity by the free carbonic acid so introduced.

Briefly, then, the coagulation and clarification of the Maumee River water require more chemical than the mere numerical turbidity values would indicate. The average amount of alumina used has been 2½ grains per gallon. With iron and lime and an unsatisfactory color reduction, 3.4 grains of iron and 1 grain of lime to the gallon were required. The reduction of color with alumina has been 77%; with iron and lime, 50%.

An average of 14,460 bacteria per c. c. was found in the unpurified water which was reduced by purification to 98 per c. c., or 99.32%. B. Coli communis was found present in 40% of 1 c. c. and 70% of 10 c. c. samples of river water, while its absence from all 10 c. c. filtered water samples has been continuous.

Although, for the most part, bacterial reduction is readily accomplished by coagulation and subsequent filtration, bleach [hypochlorite] has been used in times of high water with success. Not only has the bacterial reduction and colon removal been made more certain but a considerable saving in coagulation—due largely to the color reduction by the bleach—has been effected. The action of the bleach has not been consistently positive in either color reduction or bacterial removal. Apparently the varying organic content of the river water affects the action of

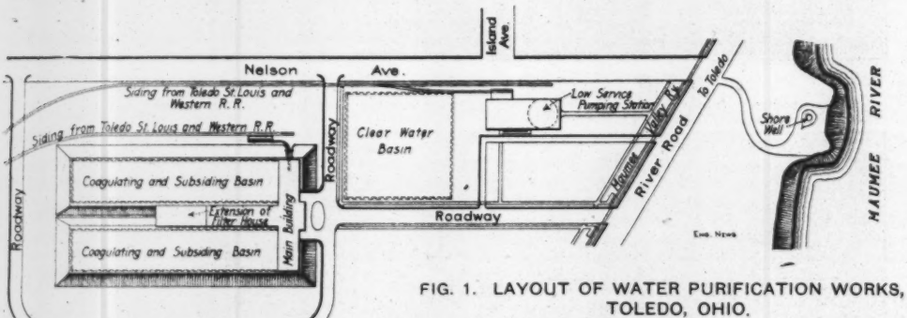

FIG. 1. LAYOUT OF WATER PURIFICATION WORKS, TOLEDO, OHIO.

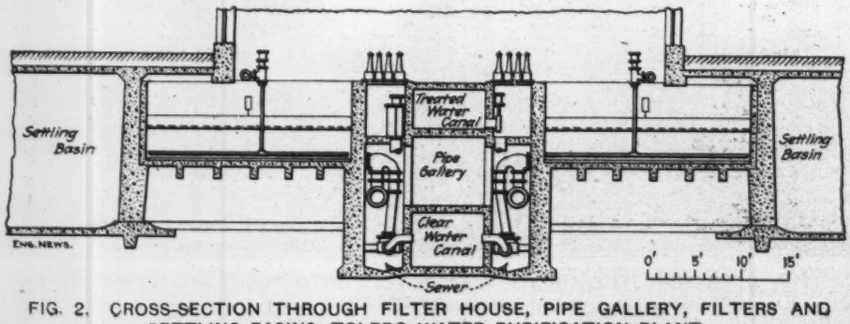

FIG. 2. CROSS-SECTION THROUGH FILTER HOUSE, PIPE GALLERY, FILTERS AND SETTLING BASINS, TOLEDO WATER PURIFICATION PLANT.

the bleach and permits aftergrowths of bacteria in the water so treated. Its continued use, therefore, is thought neither advisable nor necessary except under certain river water conditions.

Due to excessive demands upon the plant for filtered water (at times exceeding the filter capacity) a number of the units were rated at a 30% overload in the summer months. The effect of this overloading of the filters resulted in no falling off in bacterial efficiency nor did it increase the wash-water consumption. On account of the capacity of the settling basins very little work is thrown upon the filters so that increase in loss of head is slow. For best results bacterially, an arbitrary wash period is used instead of allowing the filters to reach a maximum loss of head before washing. During the year 2.2% of the water filtered has been used for washing purposes. The average daily consumption of water for the city was 15,200,000 gals., with a maximum day of 22,100,000 gals.

COSTS.—Both the pumping station and the filter plant proper are operated as one station so that it is difficult to differentiate between the two plants in some cost figures. The following figures, on a basis of cost per million gallons consumed by the city, will serve to illustrate the approximate costs during the first year of operation. For the pumping station, including supervision, labor, fuel, maintenance, repairs and supplies, $2.12 per million gallons consumed. For the filter plant proper, making charges for power furnished from the pump station and including supervision, labor, chemicals, maintenance, supplies and repairs, $4.91 per million gallons consumed. The total added cost to the city for pure water is thus $7.03 per million gallons [not including capital charges.—Ed.]. Despite this increased burden, a 10% reduction in water rates to domestic users has been recently made. This is made possible largely through the sudden increase in consumption following the introduction of pure water.

TYPHOID REDUCTION.—The availability of filtered water has undoubtedly reduced the typhoid mortality, but statistics are unreliable on account of the large number of surface wells still in use through the city. Even before the filter plant was put into service, the larger number of typhoid cases were attributed to water from shallow wells and the elimination of the typhoid or its reduction by the use of filtered water will depend largely upon the closing up of the wells known to be contaminated and a more general use of the purified city water for drinking purposes. In districts in which city water is not available, typhoid rates are far in excess of the average for the city, while in districts where city water is quite generally used, typhoid is practically absent. The only figures obtainable are somewhat incomplete but will serve to show that a reduction in typhoid through the city is at least appreciable. From January to June, 1910, there were 22 deaths from typhoid against 9 during the corresponding months this year. With a more general use of filtered water and the gradual closing up of surface wells typhoid will slowly but surely disappear.

Anti-Typhoid Inoculation has been introduced in ten hospitals in Massachusetts, led by the Massachusetts General Hospital. Altogether, 373 persons have been inoculated, the institution named leading with 153. In addition, observations have been made on 32 miscellaneous cases of anti-typhoid inoculation. This experience has been reported upon by Mark Wyman Richardson, M. D., Secretary of the State Board of Health, and Leslie H. Spooner, M. D., Boston, Mass., in a recent number of the Massachusetts Board of Health "Bulletin." The authors summarize as follows:

Our experience, therefore, covers 1,588 inoculations practiced upon 405 individuals. As yet there have been no untoward results, and we believe that the inoculated individuals have acquired an increased resistance to typhoid infection which will last them for several years at least. We expect in the coming year to extend the influence of these inoculations, especially among nurses and others attendant upon the sick. Furthermore, we have strong faith that the procedure will, within a short time, find increasing favor with the general public, which, exposed as it is to many sources of infection, is in great need of specific protection.

Secondary Stresses.

By F. C. KUNZ,* M. Am. Soc. C. E.

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In calculating the stresses in a structure, certain simplifying assumptions are made (e. g., that the joints can turn freely, and that the weight of each member is concentrated at its ends); the stresses obtained with the use of these assumptions are called the *primary stresses*. If the calculation is based on assumptions corresponding as closely as possible with actual conditions, the stresses obtained in addition to the primary stresses are called *secondary stresses*.

Nature and General Considerations.

SOURCE OF SECONDARY STRESSES.—The calculation of the primary or axial stresses in the members of a truss is made under the assumption that the members are connected to each other by frictionless hinges. In reality the members have either riveted connections so that they are unable to turn or they are pin-connected and able to turn only after the friction on the pins has been overcome. When the truss deflects under a load, the originally straight members are therefore forced to assume a curved shape which causes secondary bending stresses in addition to the uniformly distributed primary stresses. Secondary stresses in the members of a truss are further caused by the own weight and by eccentric connections of the truss members. Those due to eccentricity may be considerable, but they will not be considered here, since eccentric connections of truss members are now generally avoided.

The time required for the calculation of secondary stresses is generally overestimated. If the designer is acquainted with the method shown below he can, by a systematic arrangement, reduce the calculation for an ordinary truss to a few hours' work.

IMPORTANCE.—Until recent years, secondary stresses have been neglected, it being generally assumed that ample provision is made for them in the margin of safety of the permissible primary stresses. Recent calculations for various types of trusses have shown, however, that secondary stresses cannot be expressed by a constant percentage of the primary stresses as they vary considerably for different types. It is for this reason that more attention should be given to the secondary stresses, since of different trusses those subject to the least secondary stresses are preferable under otherwise equal conditions (strength, rigidity). The secondary stresses also depend upon the manner of loading: With a changing load they do not change in linear proportion with the primary stresses, in other words, the maximum secondary stress in a certain member does not necessarily occur simultaneously with the maximum primary stress; but generally (except in light members such as hangers, struts, etc.) the primary stress is considerably greater than the secondary stress, and therefore the *maximum total* stress usually occurs with the maximum primary stress. Further, the secondary stresses depend on the sections of the truss members, so that it is not possible to estimate them by comparison with another truss for which they are known, unless all conditions are the same.

In unusual designs, especially those of great proportions, the secondary stresses should always be calculated, even if by an approximate method only.

On the other hand, the effect of the secondary stresses should not be over-estimated, and if these stresses are below a reasonable limit they may be neglected. In the case of rolled or riveted girders in bending, we allow the same extreme-fiber stresses as for axial stresses, since the chords are concentrated as in a truss, being kept apart by the web. In bending of a compact section like a pin, however, we allow up to 50% higher stresses, which is based not only on tests with pins but also on theory, or rather the non-fulfillment of one assumption of the theory of bending, viz.: that the fibers deform independently. The fiber stresses not being uniformly distributed over the whole section, only the outer rows having the maximum stress, the less stressed adjoining fibers restrict the greater deformation and thereby the stress in the outer fibers. The assumption that a cross-section remains a plane after bending is in reality not strictly fulfilled, even for stresses within the elastic limit, and a beam is actually stronger than would seem from the theory of bending. The allowable bending stress should be derived from bending tests, but the results would vary with the shape of the cross-section.

Most specifications disregard the secondary stresses entirely, while some limit themselves to considering the stress caused by the own weight of the member, and do not take account even of this unless it exceeds a certain amount (10% of the allowed unit, according to Cooper's Specifications).

EFFECT OF YIELDING OF CONNECTIONS.—Further, it should be remembered that the conditions upon which the calculation of the secondary stresses is based are not strictly fulfilled. In riveted trusses, the members are assumed to connect rigidly at their intersection point; but the connection consists of gusset plates or at least a group of rivets, so that the rigidity will largely depend upon the size of the plates, number of rivets, etc., and the more these connections themselves can deform the more will the secondary stresses in the members be reduced. In pin-connected trusses the friction of the pins can only be assumed, and although this friction may increase in course of time it is probable that during erection, as well as afterwards through vibrations from moving loads, the members gradually turn on the pins, thus eliminating the secondary stresses at least for a part of the dead load.

EFFECT OF MANUFACTURE.—The secondary stresses depend also largely upon the methods of manufacture and erection. Secondary stresses of some amount may be caused, for instance, by incorrect shop lengths of the members, or by incorrect bevels of the faced joints, etc. If a span erected on falsework would be swung *after* all connections were riveted, the secondary stresses from dead-load would evidently be greater than if, as usual, drift pins and bolts are used before removing the falsework. A considerable influence on the secondary stresses is exerted by the method of erecting continuous chords; this is especially important in cantilever bridges.

EFFECT OF SLENDERNESS OF MEMBERS.—One of the most important factors which influence the secondary stresses is the slenderness of the members. In riveted trusses and continuous chords, the secondary stresses of a member change approximately in linear proportion with the ratio d/l (l = length, d = width of member in plane of truss); in other words, for a given length the secondary stresses are the greater the wider the member, and for a given width they are the greater the shorter the member. The least width desirable for other reasons (stiffness against buckling, simplicity of connections, etc.) should, therefore, not be exceeded. An examination of existing bridges shows that the ratio d/l, particularly of continuous compression chords, is in general greater in longer or heavier bridges; therefore, the secondary stresses in these may be expected to be higher than in short spans.

EFFECT OF PIN FRICTION.—In members hinged at both ends (eyebar chords, web members, riveted chords with pin joints) the secondary stresses cannot be greater than those caused by the friction of the pins. These stresses change, for similar sections, in linear proportion with the ratio D/d (D = diameter of pin, d = width of member) and it is therefore desirable to have the width as great as possible compared with the diameter of the pin. The following calculation illustrates the effect of the pin friction in different sections:

If S = the axial stress in member, and k = the coefficient of friction, the friction at the circumference of the pin is Sk; the moment at

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the center of the pin is $M = S\ k\ D/2$, and the bending stress caused thereby is,

$$s_t = \frac{M\,d}{2\,I} = \frac{S\,k\,D\,d}{4\,I}$$

or, making $S = s\ A$ (s = unit primary stress, A = area of section), $I = A\ r^2$ (r = radius of gyration), and $r = c\ d$ (c = constant depending on section), we get,

$$s_t = s\ \frac{k\ D}{4\ c^2\ d}\ ,$$

which shows that for a given section the bending stress caused by the friction changes in linear proportion with the primary stress.

Experiments made by Prof. Föppl have shown that k is nearly as high as 0.25. For eyebars, $c = 0.29$ and $D = 0.75\ d$ or more; we get $s_t = 0.56\ s$, that is, the friction will permit secondary stresses of generally over 56% of the primary stresses before the hinges begin to act. Such high stresses will, however, rarely occur, and eyebar members may therefore be assumed fixed at their ends.

For riveted members of box section, $c = 0.35$ and d usually not less than $3\ D$; we get $s_t = 0.17\ s$, that is, the secondary stresses cannot exceed about 17% of the primary stresses. Wide riveted members will therefore in general turn on the pins, and the maximum moments at the ends of these members may be assumed equal to the frictional moment or be entirely neglected in the calculation of the secondary stresses.

Members which are continuous over a pin, such as top chords with butt joints in simple spans, transmit to the pin only the increment of the axial stress, which is usually only a fraction of the stress itself; therefore the stress s_t caused by the friction on the pin is only a fraction of $0.17\ s$. Such members may, therefore, be assumed to turn upon the pin.

For four-angle I sections whose web plane is transverse to the truss, $c = 0.24$ and D is often as great as d, the width of the member in plane of truss. In this case $s_t = 1.1\ s$, which shows that pins can hardly be effective for such narrow members.

EFFECT OF TRUSS SYSTEM.—The published calculations of secondary stresses are limited in number, so that they do not permit definite conclusions as to the effect of the truss system on the secondary stresses. Except for the first Quebec Bridge, the longest simple span for which the calculations are published is 162 ft. These examples show, however, that in certain systems the secondary stresses are objectionably high.

It may be said, in general, that the secondary stresses in double or multiple intersection trusses are higher than in single systems. High stresses occur especially in double or multiple intersection trusses without verticals, for which reason such systems are objectionable. In single intersection systems, the distribution of secondary stresses is generally in so far favorable as they increase from center to ends of span in the chord members and from ends to center in the web members, being, therefore, highest where practical considerations usually dictate an excess of section over that required for the primary stresses. It is not always advisable to reduce the unsupported length of compression members by struts (collision struts for end posts, horizontal struts for vertical posts) as these may be the cause of high secondary stresses in those compression members. Hangers often cause high bending stresses in the bottom chord, especially if they carry great load concentrations. Usually the secondary stresses in the hangers themselves and in posts carrying only dead-load concentrations are very high; besides, these members are subjected to bending in the transverse direction due to the rigid connection of floorbeam and sway frame. It is therefore advisable to proportion these members liberally. It is evident that the secondary stresses are the greater the greater the deflection and, therefore, the shallower the truss or the higher the permissible unit stresses (as when using nickel-steel).

In continuous trusses (as used for swing bridges) and in cantilevers, the secondary stresses are highest and often very severe in the members near the intermediate supports; pins are therefore advisable at these points.

Calculation for Riveted Trusses.

ASSUMPTIONS.—Several methods of calculating the secondary stresses have been developed in recent years, especially by German engineers. The method explained here is that of Prof. Mohr (see O. Mohr, "Technische Mechanik"). It is short and easily applicable to any case.

The calculation may be made either (1) for one or two conditions of loading, as for instance full and one-sided load, or (2) with influence lines assuming successively a load unity at each panel point. By means of influence lines the absolute greatest stresses can be obtained, but the amount of work involved will seldom warrant their use. A single case of loading, preferably that causing the maximum primary stresses in most truss members (full load in simple spans), will usually suffice to give an idea of the proportion between the secondary and the primary stresses.

The calculation is based upon the assumption that the deformation of any member itself is negligible compared with that of the truss as a whole. A consideration of the deformations of the individual members would make the calculation more complicated and would affect only the stresses in very slender members in which the secondary stresses are comparatively small. In slender tension members, such as eyebars, it would result in smaller secondary stresses, since an axial pull always tends to straighten the elastic line. Further, the method of influence lines would not be applicable, since these correct stresses are higher functions of the external forces.

Other simplifying assumptions are often permissible. For instance, in the case of continuous riveted chords to which the web members are connected by pins, the influence of the web members can be neglected, as the effect of the pin friction is small as shown above. Even if the web members are riveted to the chord their effect may be neglected if their ratio I/l is less than about 1/10 of that of the chord. The calculation under this assumption is shown farther below.

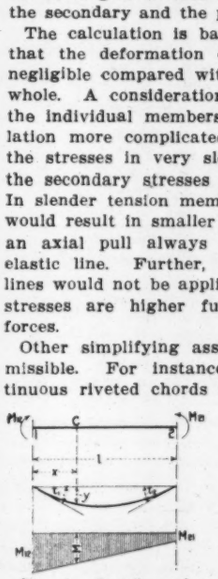


Fig. 1. Bending of a Truss Member; End Moments of Opposite Sense.

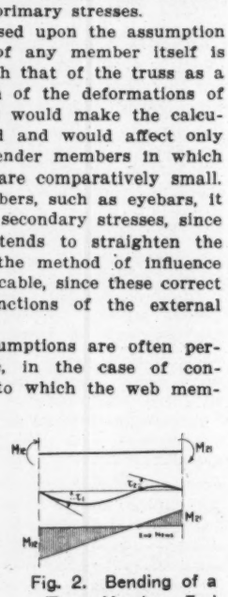


Fig. 2. Bending of a Truss Member; End Moments of Same Sense.

PRINCIPLE OF CALCULATION.—The general principle underlying all methods of calculation is the following: When the truss is loaded and deflects, any member 1-2 (Fig. 1), owing to the fixity of its ends, is subject to two moments M_{12} and M_{21}* acting on its ends, besides the axial force S. The primary stresses, produced by the force S, need not be considered here. The moments M_{12} and M_{21}, which may act in opposite sense (Fig. 1) or in the same sense (Fig. 2), cause bending moments in the member which vary along the members in straight-line ratio between the values M_{12} and M_{21} at the ends. The greatest bending or secondary stresses occur therefore at the ends and are found by the well known formula

$$s = \frac{M\,e}{I} \qquad (1)$$

where e = distance of outer fiber from neutral axis and I = moment of inertia. The main problem is, therefore, to find the moments M_{12} and M_{21}.

This can be done by applying the equation of the elastic line of the member and by the condition that the sum of all moments at a panel point must be zero.

Using the symbols τ_1 and τ_2 to denote the angles which the tangents to the elastic line of member 12 at the ends form with the straight line 1-2 (Fig. 3) the equation of the elastic line furnishes the relations

$$M_{12} = \frac{2\,E\,I}{l}\,(2\tau_1 + \tau_2);\ \text{and}\ M_{21} = \frac{2\,E\,I}{l}\,(2\tau_2 + \tau_1)$$

In these equations τ_1 and τ_2 are to be expressed in arc measure. The moments M are to be taken as positive if acting clockwise and negative if acting anti-clockwise.

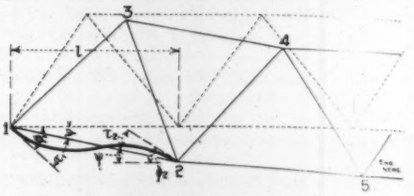


Fig. 3. Distortion of a Deflected Truss.

These formulas are obtained as follows:

The differential equation of the elastic line of the member is

$$\frac{d^2\,y}{d\,x^2} = -\frac{M}{E\,I}$$

where M is the bending moment at any point C having the coordinates x and y (Fig. 1). This moment can be expressed as follows:

$$M = M_1\,\frac{l-x}{l} - M_2\,\frac{x}{l}$$

Introducing this in the above equation and integrating, we obtain an expression for $\frac{d\,y}{d\,x} = \tan\tau$, where τ is the angle between the tangent to the elastic line at C and line 1-2. Since τ is very small, we can put τ (in arc measure) for $\tan\tau$. The constant of integration is obtained by integrating once more and considering that for $x = o$ and $x = l$, $y = o$. By introducing the values $x = o$ and $x = l$ respectively in the expression for τ we get the above formulas for τ_1 and τ_2.

MOHR'S METHOD.—Prof. Mohr makes τ_1 and τ_2 functions of the angle ψ which the straight axis 1-2 forms with the original location of this axis and the angles ϕ_1 and ϕ_2 which the original straight axis forms with the tangents to the elastic line, as follows (Fig. 3).

$$\tau_1 = \phi_1 - \psi,$$
$$\text{and}\ \tau_2 = \phi_2 - \psi.$$

Introducing these values in the above formulas for the moments M_{12} and M_{21} we get

$$\left.\begin{aligned} M_{12} &= \frac{2\,E\,I}{l}\,(2\,\phi_1 + \phi_2 - 3\,\psi) \\ M_{21} &= \frac{2\,E\,I}{l}\,(\phi_1 + 2\,\phi_2 - 3\,\psi) \end{aligned}\right\} \qquad (2)$$

Each member furnishes two such equations. The angles ψ (different for every member) are geometric functions of the axial deformations of the truss members. They can easily be found by a displacement diagram (as explained in the following example) and are, therefore, known values. Assuming the modulus of elasticity constant for all members, we can make $E = 1$ lb. per sq. in. both in the calculation of the axial deformations and in the use of Eq. (2).

The angle ϕ^1 is the same for all members at point 1 and may be called the angular deflection of panel-point 1; correspondingly, ϕ_2 is the angular deflection of panel-point 2, etc.

Each panel-point furnishes an equation of equilibrium of the form $\Sigma\ M = 0$, as, for instance, for point 2 in the sketch Fig. 3,

$$\Sigma_2\ M = M_{21} + M_{23} + M_{24} + M_{25} = 0 \qquad (3)$$

Introducing into all equations of this form the values of the moments as expressed by Eq. (2), we get a number of equations which contain only

*The first index of the moment shall always denote the panel-point at which the moment acts, while the second index denotes the panel-point at the other end of the member.

ϕ_1, ϕ_2, etc., as unknown quantities, and the values of these quantities can be obtained by solving the equations. Having the values ϕ and introducing them into Eq. (2), the moments are obtained.

COURSE OF CALCULATION.—The application of this problem comprises the following operations:

1. Calculation of the axial deformations Δl of the members due to the assumed loading.
2. Determination of angles ψ by a displacement diagram.

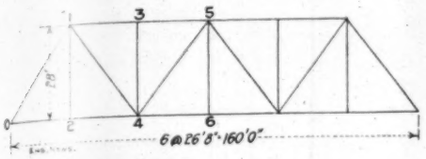


FIG. 4. SINGLE-TRACK RIVETED THROUGH PRATT-TRUSS BRIDGE, 160-FT. SPAN.
(Secondary Stresses for this truss computed in Tables I.-IV.)

3. Calculation of angles ϕ by formulas (2) and (3).
4. Calculation of moments by formulas (2).
5. Calculation of secondary stresses by formula (1).

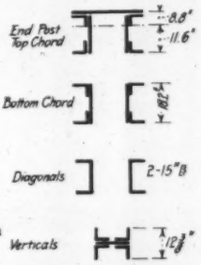
EXAMPLE.—The secondary stresses in the riveted truss of a 160-ft. single track through span (Fig. 4) will be determined. The secondary stresses from the dead-load only (1,100 lbs. per ft. per truss) will be calculated; those from a uniform live-load covering the whole span can be obtained by simple proportion.

Calculation of Axial Deformations.—The values $\Delta l = \frac{S l}{A E} = \frac{S l}{A}$ (for $E = 1$) are given for each member in Table I. The dead-load stress S is given in units of 1,000 lbs., the length l in inches, and for A we take the *gross* area of the section in square inches; therefore we get Δl in units of 1,000 ins. The calculation can be made with the slide-rule, as three numerals will suffice.

Determination of Angles ψ.—With the axial deformation Δl we draw a Williot displacement diagram (Fig. 5) by assuming point 6 and the direction of 6 — 5 as fixed. Line 6′ — 5′ represents the elongation of member 6 — 5. The displacement of point 4 with reference to point 5 consists of the movement 5′ — a in the axis of member 4 — 5 and of the movement a — 4′ at right angles thereto. Therefore, a — 4′ divided by the length l, of member 4 — 5 is the angle ψ of this member in arc measure. Correspondingly, the line drawn at right angles to the axial deformation of any other member represents the value ψl of that member. These values are given in the diagram and also in Table I.; they are scaled from the diagram with the same scale in which the axial deformations are drawn.

Careful attention should be given to the sign of ψ, which is determined as follows: For member 4 — 5, for instance, we assume point 5 immovable; the displacement diagram then shows that 4 moves from a to 4′, or clockwise around point 5 and ψ_a is given the positive sign; if 4 would move anti-clockwise around 5, ψ_a would have the negative sign. We can also assume point 4 held, then the movement of 5 at right angles to member 4 — 5 is represented by line 4′ — a, which shows that 5 turns clockwise about 4. It is therefore, immaterial which of the two points is assumed immovable, but in the displacement diagram the movement must always be considered from the immovable towards the movable point. The sign is, therefore, also independent of the point from which the diagram is started. In this example all angles ψ of the left half truss are positive.

If the truss or the loading is not symmetrical the displacement diagram has, in general, to be corrected by a parallel and a turning movement of the truss so as to conform to the conditions of the ends. A parallel movement does not change the angles ψ while a turning movement would change all these angles by the same amount ψ' equal to the angle by which the whole truss is turned, so that the corrected angles are $\psi + \psi'$. It is evident, however, that the secondary stresses are independent of a turning movement of the whole truss, being dependent only on the relative values of the angles ψ and that these need, therefore, not be corrected if they serve only the purpose of finding the secondary stresses.



Details of Members.

The values ψl are shown in units of 1,000 ins.; the angles ψ are therefore obtained in units of 1,000.

Calculation of Angles ϕ.—The factors $N = \frac{2EI}{l}$ (for $E = 1$ lb. per sq. in.) of formulas (2) are given in Table I., and also the products $N\psi$,

TABLE I.—CALCULATION OF DEFLECTION ANGLES FOR 160-FT. RIVETED TRUSS.

Member.	Length l in.	Area gross A, sq. in.	Dead load stress S, 1,000 lbs.	$\Delta l = \frac{Sl}{A}$	ψl	ψ	Mom. of inertia I,	$N = \frac{2EI^*}{l}$	ψN.
					1,000 ins.	1,000	in ins.⁴	in.-lbs.	
0 — 1	463	47.7	—101.	—980	+5,880	+12.7	3,280	14.1	+179
1 — 3	320	47.7	—112.	—751	+3,540	+11.1	3,280	20.5	+227
3 — 5	320	47.7	—112.	—751	+1,010	+ 3.16	3,280	20.5	+ 64.8
0 — 2	320	33.0	+ 69.9	+678	+5,120	+16.0	1,410	8.82	+141
2 — 4	320	33.0	+ 69.9	+678	+3,050	+ 9.52	1,410	8.82	+ 84.0
4 — 6	320	57.2	+126.	+702	+1,520	+ 4.75	1,820	11.4	+ 54.0
1 — 4	463	29.4	+ 60.8	+955	+3,940	+ 8.50	805	3.47	+ 29.5
4 — 5	463	26.5	— 20.3	—355	+1,320	+ 2.85	750	3.23	+ 9.2
1 — 2	336	19.7	+ 20.0	+341	+2,880	+ 8.58	120	0.71	+ 6.1
3 — 4	335	19.7	— 9.3	—159	+1,450	+ 4.32	120	0.71	+ 3.1
5 — 6	336	19.7	+ 20.0	+341	+ 0	+ 0	120	0.71	+ 0

*E assumed equal to 1 lb. per sq. in.

and we can now write the formulas (2) for all members (see Table II.). It is evident that, owing to symmetry of truss and loading, panel-points 5 and 6 do not turn, therefore ϕ_5 and ϕ_6 are zero.

We now write down Eq. (3) for each panel-point, introducing the above values of the moments (Table III.).

These five equations suffice for the determination of the five unknown values ϕ_0 to ϕ_4; their direct solution consumes much time, and Prof. Mohr proposes, therefore, the following approximate method by which, however, the true values ϕ can be approached as closely as desired.

From the first equation, we get a first approximation ϕ_0' of ϕ_0 by assuming ϕ_0, ϕ_1 and ϕ_2 equal to ϕ_0' and, correspondingly, from the second equation, we get ϕ_1' by assuming ϕ_0, ϕ_1, ϕ_2, ϕ_3 and ϕ_4 equal to ϕ_1', etc.

$$\phi_0' = \frac{960}{68.7} = 14.0;\ \phi_1' = \frac{1324}{116.3} = 11.4;\ \phi_2' = \frac{693}{54.9} = 12.6$$

$$\phi_3' = \frac{884}{104.6} = 8.45;\ \phi_4' = \frac{539}{68.2} = 7.90$$

Second approximations ϕ_0'', ϕ_1'', ϕ_2'', etc., we get by putting the group of equations (3) into the form given below.

Third approximations ϕ_0''', ϕ_1''', etc., are obtained by repeating the last operation, that is, by substituting in the above formulas ϕ_0'', ϕ_1'', ϕ_2'', etc., for ϕ_0', ϕ_1', ϕ_2', etc. The resulting figures are: $\phi_0''' = 14.8$, $\phi_1''' = 11.8$, $\phi_2''' = 13.4$, $\phi_3''' = 7.72$, and $\phi_4''' = 6.91$.

The values ϕ'' are very close to values ϕ''' and will as a rule suffice. Values ϕ' are not accurate enough, since the differences which appear in Eq. (2) are often small compared with the values ϕ or ψ, so that a comparatively small error in ϕ may result in a great error for M.

Moments.—The moments at the ends of the individual members are now obtained by introducing the last-found values ϕ in Eq. (2), e. g., $M_{01} = 28.2 \times 14.8 + 14.1 \times 11.8 - 537 = +45$. These moments are given in Table II. Since the

$$\left.\begin{aligned}
\phi_0'' \left(45.8 + 14.1\frac{\phi_1'}{\phi_0'} - 8.8\frac{\phi_2'}{\phi_0'}\right) &= 960 \text{ or } \phi_0 = \frac{960}{65\,2} = 14.7\\
\phi_1'' \left(14.1\frac{\phi_0'}{\phi_1'} + 77.5 + 0.7\frac{\phi_2'}{\phi_1'} + 20.5\frac{\phi_3'}{\phi_1'} + 3.5\frac{\phi_4'}{\phi_1'}\right) &= 1324 \text{ or } \phi_1'' = \frac{1324}{113\,2} = 11.7\\
\phi_2'' \left(8.8\frac{\phi_0'}{\phi_2'} + 0.7\frac{\phi_1'}{\phi_2'} + 36.6 + 8.8\frac{\phi_4'}{\phi_2'}\right) &= 693 \text{ or } \phi_2'' = \frac{693}{52.5} = 13.2\\
\phi_3'' \left(20.5\frac{\phi_1'}{\phi_3'} + 83.4 + 0.7\frac{\phi_4'}{\phi_3'}\right) &= 884 \text{ or } \phi_3'' = \frac{884}{111.8} = 7.95\\
\phi_4'' \left(3.5\frac{\phi_1'}{\phi_4'} + 8.8\frac{\phi_2'}{\phi_4'} + 0.7\frac{\phi_3'}{\phi_4'} + 55.2\right) &= 539 \text{ or } \phi_4'' = \frac{539}{75\,0} = 7.20
\end{aligned}\right\} \quad (3)$$

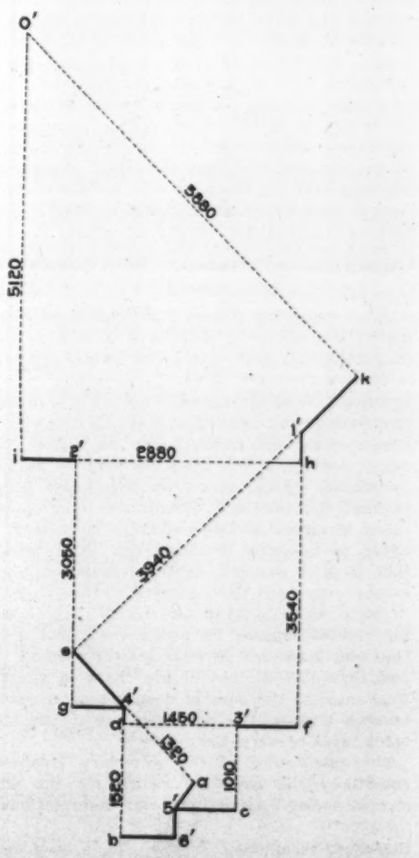


Fig. 5. Williot Displacement Diagram for 160-Ft. Riveted Truss.

values N are given in units of inch-pounds and ψ and ϕ in units of 1,000, we obtain the moments in units of 1,000 in.-lbs.

The Resulting Secondary Stresses.—These are now calculated by means of Eq. (1), $s = \frac{M e}{I}$. They are given in Table IV. in units of 1,000 lbs. per sq. in. and also in per cent. of the primary stresses. Since the section of the end post and top chord is unsymmetrical about the neutral axis, the distance e on that side of the neutral axis has to be used on which the moment causes compression. In all other members, the secondary tension and compression stresses are alike, and therefore no attention need be given to the sign of the moment in determining the secondary stress.

The relatively greatest secondary stresses occur in the verticals, since these members are very light compared with the others. In this case they are liberally proportioned for the primary stresses, so that there is no danger of their being overstressed by the secondary stresses.

Calculation for Pin-Connected Trusses.

As already explained, the friction of the pin on a member causes the bending moment $M_f = S\,k\,\frac{D}{2}$ (S = axial stress; k = coefficient of friction = 0.25 for steel; and D = diameter of pin) and the moment M caused by the distortion of the truss cannot exceed M_f, as otherwise the member would turn around the pin until M is reduced to the value M_f. In order to find the secondary stresses, we calculate first the moments M in the same manner as above explained for a riveted truss. If any of these moments M exceeds the corresponding moment M_f, we introduce the value of M_f in Eq. (3) instead of that given by Eq. (2) and solve again for the values ϕ. Introducing these new values ϕ in Eq. (2), we find a new set of moments M. This will usually suffice, but should certain moments be again considerably greater than M_f, the above calculation would have to be repeated. However, since the friction is very uncertain, it is of no value to carry the calculation too far.

Table V. shows the secondary stresses in the trusses of a 200-ft. single-track pin span (Fig. 6), calculated with an assumed pin friction of 0.18. The top chord has pin joints, except at the center where it is continuous. This accounts for the higher stress (19%) at that point compared to the stresses at other top-chord panel-points where, owing to the pins, the secondary stress cannot exceed 13% of the primary stress in the bottom fiber and 11% in the top fiber.

Approximate Calculation for a Continuous Chord.

In pin-connected trusses with continuous riveted chords (top chord of simple spans, bottom chord of cantilevers, both chords of swing spans), it is often sufficient to know the secondary stresses in these chord members, since they are comparatively wide and short. In such cases, the effect of the pin friction may be neglected, in other words the web members may be assumed to connect to the chord by frictionless hinges, so that the influence of the web and opposite chord members on the secondary stresses of the chord in question is eliminated. The work is then greatly reduced, as the calculation has to extend only over the members of that chord.

The same simplifying assumption may be made for riveted trusses in which the value I/l of the web members is comparatively small, say less than 10% of that of the chord in question. This may be the case in deep simple trusses or in arch trusses, the web members in the latter being usually very light.

The calculation of the secondary stresses is essentially the same as shown for the whole riveted truss. After the axial deformations of the chord members $\Delta\, l = \frac{S\, l}{A\, E}$ have been determined (assuming $E = 1$), the angles ψ may be obtained either by a displacement diagram as already shown, or by the following method, which is often shorter:

The vertical deflections of the panel-points along the chord in question are first determined by any convenient method. If for any member 1 — 2 in Fig. 7, the ordinates of the deflection polygon representing the deflections of points 1 and 2 are called y_1 and y_2, and λ is the panel length, the angle of deflection is

$$\psi = \frac{(y_2 - y_1)\lambda}{l^2}$$

or, if the member is nearly horizontal,

$$\psi = \frac{y_2 - y_1}{\lambda}$$

If 2 is the right-hand panel-point, ψ is positive when $y_2 > y_1$ and negative when $y_2 < y_1$. Having the values ψ and $N = \frac{2\,I\,E}{l}$ for all chord members, we can substitute them in equations (2) and (3) and from these obtain the values ϕ and M in the manner shown for riveted trusses. Equations (3) are here very simple, since at each panel-point there are only two moments, which must be equal and of opposite sign.

Secondary Stresses from Own Weight of Members.

The stresses in each member due to its bending under its own weight are usually small, and it probably corresponds closely to actual conditions if the members are assumed to act as simple beams, since they act as such before the end connections are riveted and will approximately remain in this condition afterwards. The maximum moment occurs, therefore, at the center and is $M_w = \frac{W\,l'}{8}$ where W = weight of member and l' the horizontal distance between its ends. The moments at the ends are zero. The center moment M_w causes the bending stress $s_w = \frac{M_w\, e}{I}$. If we want to combine this stress with that due to the deflection of the truss, it will suffice to figure the latter at the center and add both stresses. If s_1 and s_2 are the secondary stresses due to the distortion of the truss at the ends of the member (with their proper sign) the stress at the center is $s = \frac{s_1 + s_2}{2}$, which has to be added algebraically to s_w. See Fig. 8.

Care should be taken to combine only stresses of the same extreme fiber (upper or lower) and

TABLE II.—NUMERICAL VALUES OF EQ. (2) FOR MEMBERS OF 160-FT RIVETED TRUSS.

MEMBER.		
0 — 1	$M_{01} = 28.2\,\phi_0 + 14.1\,\phi_1 - 537$	$M_{10} = 14.1\,\phi_0 + 28.2\,\phi_1 - 537$
1 — 3	$M_{13} = 41.0\,\phi_1 + 20.5\,\phi_3 - 681$	$M_{31} = 20.5\,\phi_1 + 41.\,\phi_3 - 681$
3 — 5	$M_{35} = 41.0\,\phi_3 - 194$	$M_{53} = 20.5\,\phi_3 - 194$
0 — 2	$M_{02} = 17.6\,\phi_0 + 8.8\,\phi_2 - 423$	$M_{20} = 8.8\,\phi_0 + 17.6\,\phi_2 - 423$
2 — 4	$M_{24} = 17.6\,\phi_2 + 8.8\,\phi_4 - 252$	$M_{42} = 8.8\,\phi_2 + 17.6\,\phi_4 - 252$
4 — 6	$M_{46} = 22.8\,\phi_4 - 162$	$M_{64} = 11.4\,\phi_4 - 162$
1 — 4	$M_{14} = 6.9\,\phi_1 + 3.5\,\phi_4 - 88$	$M_{41} = 3.5\,\phi_1 + 6.9\,\phi_4 - 88$
4 — 5	$M_{45} = 6.5\,\phi_4 - 28$	$M_{54} = 3.2\,\phi_4 - 28$
1 — 2	$M_{12} = 1.4\,\phi_1 + 0.7\,\phi_2 - 18$	$M_{21} = 0.7\,\phi_1 + 1.4\,\phi_2 - 18$
3 — 4	$M_{34} = 1.4\,\phi_3 + 0.7\,\phi_4 - 9$	$M_{43} = 0.7\,\phi_3 + 1.4\,\phi_4 - 9$
5 — 6	$M_{56} = 0$	$M_{65} = 0$

TABLE III.—NUMERICAL VALUES OF EQ. (3) FOR PANEL POINTS OF 160-FT. RIVETED TRUSS.

Panel Point.		
0	$M_{01} + M_{02} = 45.8\,\phi_0 + 14.1\,\phi_1 + 8.8\,\phi_2 - 960 = 0$	
1	$M_{10} + M_{12} + M_{14} + M_{13} = 14.1\,\phi_0 + 77.5\,\phi_1 + 0.7\,\phi_2 + 20.5\,\phi_3 + 3.5\,\phi_4 - 1134 = 0$	
2	$M_{20} + M_{24} + M_{21} = 8.8\,\phi_0 + 0.7\,\phi_1 + 36.6\,\phi_2 + 8.8\,\phi_4 - 693 = 0$	
3	$M_{31} + M_{34} + M_{35} = 20.5\,\phi_1 + 83.4\,\phi_3 + 0.7\,\phi_4 - 884 = 0$	
4	$M_{42} + M_{41} + M_{43} + M_{46} = 3.5\,\phi_1 + 8.8\,\phi_2 + 0.7\,\phi_3 + 55.2\,\phi_4 - 539 = 0$	

TABLE IV.—END MOMENTS AND RESULTING SECONDARY STRESSES IN MEMBERS OF 160-FT. RIVETED TRUSS.

Member.	Panel-point.	Moment, in 1,000 in.-lbs.	Moment of inertia I, in ins.⁴	Extreme fiber distance e, in ins.	Primary stress, in 1,000 lbs. per sq. in.	Secondary Stresses: In units of 1,000 lbs. per sq. in.	Secondary Stresses: In percent. of primary stress.
0 — 1	0	+ 45	3,280	8.8	— 2.12	— 0.12	6
	1	+ 3	3,280	11.6	— 2.12	— 0.01	1
1 — 3	1	— 40	3,280	11.6	— 2.35	— 0.14	6
	3	—123	3,280	8.8	— 2.35	— 0.33	14
3 — 5	3	+123	3,280	8.8	— 2.35	— 0.33	14
	5	— 36	3,280	8.8	— 2.35	— 0.10	4
0 — 2	0	— 45	1,410	9.1	+ 2.12	+ 0.29	14
	2	— 56	1,410	9.1	+ 2.12	+ 0.36	17
2 — 4	2	+ 45	1,410	9.1	+ 2.12	+ 0.29	14
	4	— 12	1,410	9.1	+ 2.12	+ 0.08	4
4 — 6	4	— 4	1,820	9.1	+ 2.20	+ 0.02	1
	6	— 83	1,820	9.1	+ 2.20	+ 0.41	19
1 — 4	1	+ 17	805	7.5	+ 2.07	+ 0.16	8
	4	+ 1	805	7.5	+ 2.07	+ 0.01	1
4 — 5	4	+ 17	750	7.5	— 0.77	— 0.17	22
	5	— 6	750	7.5	— 0.77	— 0.06	8
1 — 2	1	+ 8	120	6.2	+ 1.02	+ 0.41	40
	2	+ 9	120	6.2	+ 1.02	+ 0.46	45
3 — 4	3	+ 7	120	6.2	— 0.47	— 0.36	77
	4	+ 7	120	6.2	— 0.47	— 0.36	77

TABLE V.—Secondary Stresses in 200-Ft. Single-Track-Pin-Connected Span.

Member.	Panel-point.	Primary stress, in 1,000 lbs. per sq. in.	Secondary Stress: In 1,000 lbs. per sq. in.	Secondary Stress: In % of primary stress.
Bottom Chord.				
0 — 1	0	+ 2.48	± 0.29	12
	1		± 0.44	18
1 — 2	1	+ 2.48	± 0.38	15
	2		± 0.07	3
2 — 3	2	+ 3.22	± 0.17	5.5
	3		± 0.21	6.5
3 — 4	3	+ 3.22	± 0.04	1.2
	4		± 0.21	6.5
End Post.				
0 — 5	0	— 2.40	— 0.12	5
	5		— 0.18	7.5
5 — 6	5	— 2.76	— 0.25	9
	6		— 0.20	7.5
Top Chord.				
6 — 7	6	— 2.74	— 0.22	8
	7		— 0.15	5.5
7 — 8	7	— 2.78	— 0.11	4
	8		— 0.53	19
Diagonals.				
5 — 2	5	+ 2.78	± 0.16	6
	2		± 0.05	2
6 — 3	6	+ 2.15	± 0.09	4
	3		± 0.01	0.5
7 — 4	7	+ 1.27	± 0.10	8
	4		± 0	0
Verticals.				
5 — 1	5	+ 1.52	± 0.56	37
	1		± 0.60	40
6 — 2	6	— 1.53	± 0.13	8.5
	2		± 0.23	15
7 — 3	7	— 0.76	± 0.11	15
	3		± 0.11	15
8 — 4	8	— 0.51	...	..
	4		...	..

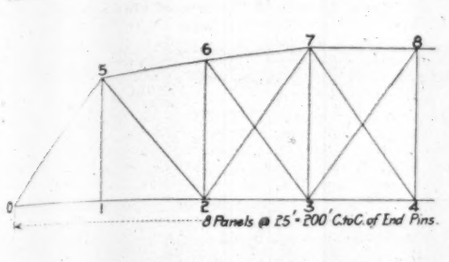


FIG. 6. SINGLE-TRACK PIN-CONNECTED THROUGH TRUSS BRIDGE, 200-FT. SPAN.

(Secondary Stresses for this truss given in Table V.)

the combined stress must be considered only if it is greater than either s_1 or s_2 alone.

Secondary Stresses Due to Other Causes.

There are other secondary stresses in the members of the trusses and in almost every other part of a bridge. It would have no practical value to calculate all these stresses, but they should be considered in the design with a view of minimizing them as far as possible or providing ample material where they cannot be avoided and are liable to be excessive. They should not be avoided at the expense of rigidity.

Following the path of transmission of the loads, from the flooring to the supports, we find first in most cases that the load is not applied centrally to the girders or stringers as assumed. The eccentricity causes a torsional moment which in some cases has been found to produce considerable stress. To minimize this the stringers should be effectively stayed laterally by top laterals and by additional cross-frames if they are long.

The stringers are analyzed as simple beams, while in fact they are continuous beams over elastic supports (floorbeams) at which considerable negative moments occur. The stringers are usually sufficiently strong for these moments, since their flange angles are of full length, but,

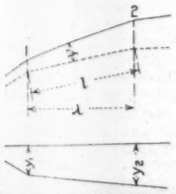


Fig. 7. Direct Calculation of Deflection Angles from Truss Deflections.

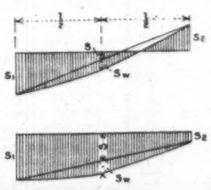


Fig. 8. Addition of Distortion and Sagging Stresses in a Member.

in the case where the stringers are framed into the floorbeams, the latter may under one-sided load be subject to high torsional stresses and thereby cause bending stresses in the verticals to which they are connected. These stresses can be reduced by using deep stringers which have small deflections.

The floorbeams are calculated as simple beams, while they act as partially fixed at their ends. Investigations show that the fixity is in fact small, that is, the floorbeams act nearly as simple beams, on account of the distortion of the cross-section of the bridge. This, however, means that bending stresses occur in the verticals which form part of the frame. To minimize these the floorbeams should be made deep and the verticals of through spans comparatively slender in the transverse direction.

This distortion of the cross-section may be the cause of considerable secondary stresses in the top flange of through plate-girders or the top chord of pony trusses, which is forced to deflect laterally more at the loaded than at the unloaded panel-points.

A frequent source of secondary stress in the trusses is the longitudinal friction of their movable ends under the live-load or under temperature changes. This friction may be minimized by preventing the collection of dirt around the rollers. The friction is the smaller the greater the diameter of the rollers. In recent specifications, the minimum permissible diameter is 6 ins.

Direct Stress Measurements.

Measurements of stresses in existing bridges form a valuable supplement to the calculation of the secondary stresses, since, if carefully and rationally made, they give a more reliable picture of the stresses as they actually exist. Such measurements also form a check of the reliability of the calculations.

Stress measurements have been made systematically first on bridges of the Orléans Railway in France (1897 to 1899); these measurements were described by M. Lanne at the International Congress for Testing Materials in Paris in 1900. The main principles involved were described by M. Rabut in a paper before the "Conference for Experiments on Bridges" (see "Annales des Ponts et Chaussées," 1901).

Stress measurements are based on the laws of elasticity and their application is, therefore, confined to stresses within the limit of proportionality. When the change Δd in the length d of a fiber is measured, the unit stress s causing this change is

$$s = \frac{\Delta d}{d} E$$

provided the modulus of elasticity E of the material is known. Since a certain minimum length d is required for the measurement, it follows that the stress cannot be measured exactly at a certain point, but only the average stress in the distance d. This is, however, not important where d is small compared with the total length of the member. It is also evident that dead-load stresses can only be measured directly at the moment when the member is stressed from zero up to the full dead-load stress; for instance, in bridges erected on falsework at the moment when the latter is released.

In existing structures, only stresses from superimposed loads can be measured. The distortions of the fibers are measured by extensometers, several types of which are in the market. The best known are those of Turneaure, Henning, Fränkel-Leuner, Dupuy, Manet-Rabut (Annales des Ponts et Chaussées, 1896), etc. Most of these not only indicate the maximum stress, but register a continuous stress line for a load moving over the bridge, including vibrations and impact.

In order to get a complete analysis of stresses in a truss member $a - b$, Fig. 9, simultaneous measurements at six points of the member at least are necessary; three instruments to be attached at points 1, 2, 3 of section I. (A & B, Fig. 9) and three at the corresponding points of section II.

The three points of a section should be as far apart as possible and on different sides of the neutral axis. The two sections I. and II. should be as near as possible to the gusset-plates, where the secondary stresses are the greatest, and should be selected so that there is no panel-point between them. Since, according to the usual assumption in the theory of flexure, cross-sectional planes remain plane after bending, the elongations and therefore the stresses in all fibers of section I. can be determined from the three measured elongations. Further, since the stress in a fiber changes in a straight line from section I. to II., the stress k at any intermediate point of that fiber is obtained by simple proportion (as in sketch C in Fig. 9) from the stresses k_1 and k_2 at I. and II. Thus the stress at any point of the member can easily be found.

In order to have a check measurement, it is well to attach an instrument at a fourth point (4) of each section. For flat bars, two instruments at each section will suffice.

Measurements made with only one or two instruments attached to a member are insufficient since only one or two local stresses are thus registered, which may not be the greatest.

In the case of a bridge having been riveted complete before it carried its own weight, the dead-load stresses can be measured indirectly at any time. M. Rabut proposes two methods: One method requires a test load distributed similarly to the dead-load. The stresses measured under the test load multiplied by the proportion of dead-load to test load, give the actual dead-load stresses. The other method uses a concentrated moving load, the effects of which are measured with the load placed successively at each panel point, thus furnishing the influence line from which the effect of the dead-load concentrations can be found by proportion. This assumes that the secondary stresses change in linear proportion with the load, which is not quite correct but sufficiently close for practical purposes.

Since the effect of a change of temperature on the results is considerable, the measurements should be made at a time when the temperature remains nearly constant, preferably in the morning before sunrise. In any case, the instruments should be protected from the sun, and the test made in as short a time as possible.

Stress measurements on other parts of a bridge (laterals, floor, etc.) are made in a way similar to that described for truss members.

A brief summary of the results of the measurements on the Orléans Railway bridges may be of interest.

The experiments were chiefly made to test certain short-span plate-girder bridges whose calculated stresses were excessive. The measured stresses were considerably lower.

Plate-girders supporting on their top flange longitudinal ties and the rails, acted more or less as beams with fixed ends, showing stresses

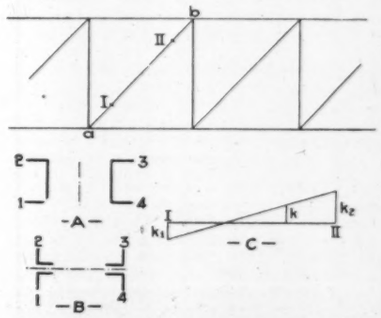


Fig. 9. Required Strain Measurements for Determining Actual Stresses.

at the center less than half the theoretical stresses, and at the ends compression in the bottom flange and tension in the top flange. This was especially the case where no rail joint was located on the span.

The stresses in truss members were found to vary considerably over a cross-section. In pony bridges, the greatest chord stress occurred on the outside of the top chords. This can be explained by the fact that when the greatest bending moment in the truss is produced, the floorbeams near the center deflect more than the end floorbeams, thus causing the top chords to bend inward near the center of the bridge. The ratio of the stresses at the inside face to those at the outside face was as low as 0.62.

The bottom chords of through bridges always showed smaller stresses than the top chords; the stringers and the bottom laterals took part in the elongation, relieving to some extent the bottom chords.

Floorbeams were found to act as simple beams, that is, without fixity at their ends. This was also the case with the stringers for a load on one side of the floorbeam; for symmetrical loading the stringers acted as beams with fixed ends.

In a Pratt-truss bridge 180 ft. long, carrying a single track (see M. Mesnager, Annales des Ponts et Chausées, 1899), the secondary stresses in the verticals reached 200% of the direct stresses, reversing on one side their compression to tension,

while those in the diagonals reached 45% of the direct stresses. These large percentages may be ascribed to the peculiar design of the trusses and should not be considered for general conclusions.

The average stresses agreed fairly well with the calculated primary stresses if the latter were figured by using the gross area in tension as well as in compression.

Every bridge, even if of conventional design, will act more or less individually concerning secondary stresses, which in itself is sufficient reason for determining its safety whenever practicable by means of stress measurements.

INDIRECT MEASUREMENT OF SECONDARY STRESSES.—Mr. W. Gehler in his book "Nebenspannungen eiserner Fachwerkbrücken" (Berlin, 1910) describes an indirect method which was used for measuring the secondary stresses in a single-track through-truss span of the Pratt type on the line between Dresden and Elsterwerda. As stated above, the direct measurements depend upon the accuracy of the instruments and are, therefore, not always reliable, especially in short members. In the above experiment, the secondary stresses in the loaded chord were determined as follows:

The deflections of the lower panel-points were measured by contact levers. These deflections allowed the calculation of the angles ψ (as shown before) in Prof. Mohr's method of calculation of the secondary stresses. Further, the angular deflections ϕ of the lower panel-points were measured by levels placed on top of the bottom chord at each panel-point. The bending moments were then obtained directly by introducing the values of ψ and ϕ in Eq. (2). The results obtained agreed satisfactorily with those of original calculation.

Quadruple Brake Equipment is provided on electric cars of the Bernina railway, a new 37-mile transit line in the Alps connecting St. Moritz, the southern terminus of the Albula railway in Switzerland, with Tirano, on the Valtellina railway in Italy. The maximum gradient is 7%, and there are many heavy grades of great length. For safety on these grades, the cars have hand brakes, Hardy vacuum brakes, electromagnetic track brakes, and provisions for electric braking by short-circuiting the motors.

Four-Cylinder Simple Locomotives of the 4-6-0 class are being introduced by the London & Southwestern Ry. (England) for handling heavy passenger trains on certain runs where it became desirable to reduce the time between terminals. The four cylinders are all in a line at the smokebox, but the inside cylinders are connected to the first driving axle while the outside cylinders are connected to the second axle. Each cylinder is served by a piston valve directly above it, but only two sets of valve gear are used; these are of the Walschaerts type, operated from the outside driving cranks, the inside valve being driven from a horizontal rocker connecting the two valve stems. The special equipment includes transverse water tubes in the firebox, a tubular steam drier (not superheater) in the smokebox, a feedwater heating system and duplex pumps for the feed-water and a steam reversing-gear. The engines are notable also in having the tenders mounted each on a pair of trucks, six-wheeled tenders being usually adopted on English railways. The engines were designed by Mr. Dugald Drummond, Superintendent of Motive Power, and were built at the railway company's shops. The principal dimensions are as follows:

Driving wheels (6)	6 ft. 7 ins.
Truck and tender wheels	3 ft. 7 ins.
Wheelbase, driving	14 ft. 4 ins.
Truck	6 ft. 6 ins.
Engine	27 ft. 7 ins.
Engine and tender	53 ft. 5 ins.
Total length; engine and tender	63 ft. 0 ins.
Weight on drivers	123,088 lbs.
On truck	47,824 lbs.
Engine, total	170,912 lbs.
Engine and tender	269,472 lbs.
Boiler, diameter, inside	4 ft. 9¼ ins.
Working steam pressure	200 lbs.
Firebox; Belpaire type	100 × 42 ins.
Tubes (Steel); No., 247; diam. (outside)	1¾ ins.
Length	13 ft. 9 ins.
Firebox water tubes (copper); No. 84; diameter (outside)	2¾ ins.
Length (over headers)	5 ft. 6 ins.
Cylinders (4)	15 × 26 ins.
Connecting rods, length	6 ft. 6 ins. & 11 ft. 0 ins.
Height, to cen. line of boiler	9 ft. 3½ ins.
To top of smoke stack	13 ft. 2½ ins.
Water in tender	4,500 Imp. galls.

Service Equipment of the Pennsylvania Tunnels and Terminal, New York City.*

By GEORGE GIBBS,† M. Am. Soc. C. E.

Terminal Yards.

The main approach tracks from the west, fan out, from the tunnel portals at Tenth Ave. into six running tracks, three for each direction, to the main switch leads at Ninth Ave., and thence by double ladders north and south into 21 platform tracks covering the full width of the yard. At the east end of the station yard all except five of these tracks are again gathered into two groups leading to the crosstown tunnels. At the extreme southeast of the yard the five stub tracks end at Seventh Ave., to lead in the future to two tunnels under 31st St. It has been the aim to provide double leads throughout as far as possible so that simultaneous parallel movements can be made in reaching the various tracks connected therewith.

The station yard is not intended to provide unlimited storage, but considerable space is available in the corners formed by the rectangular configuration of the lot and the rectangular extension west of Ninth Ave.

The parallel tracks are spaced at 31-ft. centers generally where platforms are between and at 15-ft. centers otherwise to allow for building supports. The total length of tracks in the station yard is about 16 miles, of which three are adjacent to platforms.

SUBWAYS.—In the terminal yard between Seventh and Ninth Aves a comprehensive system of under-track subways has been constructed (1) for housing the various piping systems to the buildings, (2) to facilitate trucking baggage, mail and express matter from one platform to another, and (3) for communicating between the various important building units, such as the main station, post-office and express building.

In the portion of the yard under and west of the most important signal cabin at the intersection of the main-track ladder system, accessible space was required for the large quantity of signal apparatus. This was secured by a basement for the building below the tracks and a longitudinal subway for conduits and instruments from which all connections were conveniently made to switches and all apparatus was easily accessible. This system comprises, therefore, longitudinal and cross subways, which furnish an expensive, but valuable and comprehensive means of intercommunication without interfering with the scheme of tracks or train operation, or affecting the building materially.

The subways comprise a main longitudinal system from Seventh to Ninth Aves., used in part for pipes, in part for trucking and generally for both. This system is intersected by cross galleries consisting of baggage-trucking subways at each end of the Long Island R. R. platforms on the north side of the yard (for local suburban traffic), a main cross subway immediately under the outgoing baggage-room in the station building and communicating with all lifts, and four cross pipe subways, 400 ft. apart, to cover the whole yard.

By this system, all platforms have intercommunication through the subways and lifts. The station building has access to the post-office and will have access to the express building when that is erected by a subway and lift to the operating floors.

Station Building.

With the advent of electric-propelled trains, it is entirely possible to utilize the basement of the building for tracks and the levels immediately above for passenger facilities.

When passengers arrive at the station, they are nearest to the train. This involves different levels and the use of stairs or lifts, but in order that a station may be in the front of a great city depression or elevation of the tracks is unavoidable and stairs are a necessity. The form of construction adopted for the Pennsylvania Station therefore is that of a bridge over the yard and platforms, the building having its main floor intermediate in level between the streets and the track platforms.

The building may be entered from any one of its four sides, so that the distances, which are necessarily considerable in a station of large capacity, are the least possible. It is believed that the unusual opportunities for entrance and exit at the centers of the four sides and the separation of incoming and outgoing passengers on different levels make this building unique among the large stations of the world in the distribution of crowds without delay or confusion.

CONSTRUCTION DATA.—The following is a list of the construction dates, and quantities of the more important materials used in the building:

Building foundations begun	June 1, 1906	
First steel column erected	May 27, 1907	
First stone of masonry set	June 15, 1908	
Finished exterior masonry	July 31, 1909	
Building substantially completed	Aug. 1, 1910	
Maximum number of men employed	4,240	
Average number of men employed	1,800	
Granite, exterior	490,000	cu. ft.
Granite, miscellaneous and in concourse	60,000	" "
Marble, interior	24,000	" "
Travertine	71,589	" "
Artificial stone	11,600	sq. yd.
Concrete fireproofing, cinder	243,000	cu. ft.
Concrete fireproofing, stone	720,000	" "
Granolithic floors	310,000	sq. ft.
Marble floors	85,000	" "
Cork floors	11,000	" "
Maple floors	147,000	" "
Terrazza floors	10,000	" "
Vault lights	155,000	" "
Brickwork, all kinds	17,000,000	brick
Terra cotta furring and partitions	600,000	sq. ft.
Roofing, metal	300,000	" "
Roofing, tile	150,000	" "
Roofing, skylights	83,000	" "
Structural steel	27,000	tons
Ornamental iron	2,500	"
Glazing	80,000	sq. ft.
Plastering	85,400	" "
Painting (area)	2,800,000	" "
Cement	64,000	bbls.

Operating Arrangement.

The tracks are arranged for through and stub-end operation, but for the majority of movements it may be considered a stub-end station. Through trains on both the Pennsylvania and Long Island R. R.'s are hauled by electric locomotives; and suburban trains on the latter by multiple-unit motor cars. There are eleven platforms serving 21 tracks. Nos. 1 to 16 on the south side are normally assigned to the Pennsylvania service;

FIG. 1. A PIPE SUBWAY UNDER TRACKS; PENNSYLVANIA STATION, NEW YORK CITY.

*Abstract of a paper to be presented before the American Society of Civil Engineers, Oct. 18, 1911, entitled "The New York Tunnel Extension of the Pennsylvania R. R.; Station Construction, Road, Track, Yard Equipment, Electric Traction and Locomotives," printed in the May Proceedings, p. 636 and following.

[For the construction features of this project the reader should consult articles in our previous issues, of Dec. 13, 1906; Feb. 28, 1907; Sept. 15, 1910. The electric locomotives were described Nov. 11, 1909. —Ed.]

†Chief Engineer of Electric Traction and Station Construction, Pennsylvania Tunnel & Terminal R. R. Co., Pennsylvania Station, Seventh Ave. and 32d St., New York City.

FIG. 2. PIPING AND HOT AIR DUCTS IN STATION BASEMENT.

FIG. 3. MAIL HANDLING MACHINERY UNDER POST OFFICE; PENNSYLVANIA TERMINAL, NEW YORK CITY.

the remaining five tracks to the north are assigned to the Long Island.

Pennsylvania trains, except certain short-distance expresses from Philadelphia and certain locals, after unloading, proceed through to the terminal yard in Long Island City, where they are made ready for the return trip. Short-distance expresses are turned and stored in the station yard, and all Long Island trains are cared for in the station yard. The 33d St. tunnels are normally used for the Long Island traffic, but the 32d St. tunnels may be used in emergencies, making all station tracks, except the southernmost four, available for Long Island R. R. service.

PLATFORMS.—A departure from the usual American practice of making track platforms about 9 ins. high above the rails was decided upon in making the platforms flush with the floor of the car. This gave (1) greater ease and saving in time in loading and unloading cars, (2) a saving of about 4 ft. in the vertical lift between platforms and street, (3) the elimination of the dangerous practice of crossing tracks at grade and (4) gave space under the platforms for machinery and piping for hydraulic elevators and space for housing signal and other electrical apparatus. The new steel cars used for all purposes have their vestibules so arranged that the side doors may be opened without lifting the trap over the steps. The fascia over the doors is high enough to permit passengers walking out at the platform level. The difficulty of handling baggage was minimized by using a special truck with a platform only 9 ins. above the car floor. The first cost of the high platforms was not excessive, because of the cheapening of other constructive features.

The eleven passenger platforms vary in width from 20 to 40 ft. and in length from 750 to 1,170 ft. All are island platforms. In addition, there are island platforms west of the station proper and under the post-office, which are used for mail purposes exclusively. Space has been assigned for two additional platforms at the south side of the yard near Ninth Ave. for the future express building.

Elevators.

Elevator service had to be provided for: (1) passengers from track platforms; (2) baggage between baggage-room, platforms and subways; (3) office service from street level to floors above; (4) between the restaurants and the kitchens; (5) escalators from the concourse to the street level, and (6) facilities for handling mail from trucking subways to the main floor of the post-office.

Each platform was given a passenger elevator between the train level and the exit concourse. It was not possible to operate these to the street level nor to make them of sufficient capacity to handle the entire train loads.

Each of the nine long station platforms has been equipped with two baggage lifts; one from the outbound baggage-room on the west side of the building, and one from the inbound baggage-room on the east side. The two short Long Island platforms, on the north side of the station, have one lift each. Mail handled into the post-office required the installation of four lifts.

Escalators from train platforms to the concourse were considered, and space was provided for them in the building construction, but it was decided to defer their installation until their necessity was demonstrated. If not needed, they would be objectionable, as they would transfer passengers to the main instead of the exit concourse, causing a conflict of passenger movement which would defeat the chief advantage of the double concourse system.

A special condition prevails on the north side of the station, where a heavy commuter traffic is cared for. Here an escalator leads passengers from the exit concourse, under 33d St., to the street level and lands them without effort in a private street near the 34th St. crosstown surface car line.

The distribution of the elevator and lift system over about 20 acres indicated that electric distribution of power rather than hydraulic would be simpler and cheaper. In the case of the baggage lifts, however, it was desired to secure the advantage of the hydraulic plunger type because of ease and accuracy in control and simplicity. The concourse passenger elevators were similarly equipped, placing all piping and apparatus in the space under the platforms.

The conditions were different with the office elevators and kitchen dumb-waiters, where the plungers would interfere with the clearances over the tracks. Here electric elevators were adopted. Power for hydraulic lifts is furnished by pumps in the service plant, and current for the electric elevators is taken from the general traction-power mains through a special switchboard connection.

The electric elevators, being over the platforms and tracks, required special safety precautions. Therefore, all shafts were provided with air-cushion wells and arresters. Thorough tests were made of the effectiveness of all these devices. For instance, a fully loaded car was cut loose at the top of a shaft and allowed to drop freely 70 ft. into the cushion, and was brought to a stop without spilling water or breaking an egg.

To provide against the possibility of a baggage truck breaking through a gate at the platform level, collision-proof gates of steel plates and angles have been used.

In order to obviate the necessity of an attendant at each lift, electric control of the starting mechanism was installed. This consists of (1) push buttons located at each landing and on the car itself; (2) electric circuits to a control board, and (3) an electrically-operated pilot valve on the board controlling the hydraulic valve mechanism. A car can be called for or sent to any landing automatically by pressing the button for that landing. The car cannot be operated by the push-button control if any gate is open, and when a car is at rest and the gate is open the operating rope is locked against movement until the gate is closed. In addition to the automatic control, the lifts may be operated in the usual way by hand ropes.

Baggage Handling.

These facilities have been planned for as rapid service as consistent with large platform area, and to avoid long-distance trucking on platforms. Two baggage-rooms have been provided, one at the east side adjoining the main waiting-room and the driveways for delivering to and from wagons, for checking, and for baggage arriving from trains. The other is on the west side adjoining Eighth Ave. for delivery to trains. The east room contains all the usual facilities for weighing, checking, storage, etc., and is the one to which the public has access. The west room is chiefly a passageway for reaching various platform lifts. The two rooms have communication by a trucking passageway under 31st St. and under the Seventh Ave. front of the building. The following data relate to baggage handling:

Estimated quantity of baggage handled per hour	220	pieces
Total quantity per 24 hours	6,025	"
Storage capacity	6,000	"
Length of run from east to west room	1,350	ft.
Time required to deliver truck load to car at west end of platform	8	min.
Number of baggage scales	4	
Capacity, total	20,000	lbs.
Capacity, weighing	10,000	"

There are four kinds of baggage trucks for station uses. The special trucks for general baggage purposes were designed and built by the Motive Power Department, and are similar to those in use elsewhere, except that they have drop frames for greater convenience. These are either self-propelling or trailing.

TRAIN INDICATORS.—In the main concourse at the head of the stairs to each car platform, are gates and illuminated signs. The latter consist of cast-iron columns 16 ft. high, two at each gate. The top of each is four-sided and contains mechanism for moving steel tapes opposite the openings on the four sides simultaneously, on which tapes the numerals 1 to 10 are enameled. The mechanism is operated by a crank key through vertical rods and gearing. By turning this key the tapes may be set to indicate any desired departure time. The posts also carry a four-sided card box, in which destination signs are shown.

TRAIN-STARTING SYSTEM.—It was necessary to have quick communication between the gateman, the conductor of the outgoing train and the train director in the signal cabin to ensure prompt starting of trains at schedule time. At the head of the stairs leading to the platform there is a push-button and lamp indicator, and at four points on the platform a somewhat similar instrument. All these various devices are interconnected and operate as follows. About one minute before the train is to leave, the conductor inserts a key in the train director's instrument, showing the number of the train for which the announcement is given. The director then moves a lever closing a circuit and lighting the lamp in the conductor's box and at the platform gate, to indicate to both conductor and gateman that the route has been set for the train. When a gateman closes his gate at the leaving time, he pushes a button extinguishing the indicating light, and at the same time by lighting a lamp at the platform signal stations notifies the conductor that the gate is closed. When all passengers are aboard the conductor operates a push-button circuit-breaker, extinguishing all lights and restoring the apparatus to normal.

FIG. 4. TRAIN STARTING SIGNAL AND TELEPHONE SET AT PLATFORM LEVEL.

CLOCK SYSTEM.—Electrically-operated and centrally-synchronized clocks have been provided throughout the station and yard. There are four 6-ft. dials on the facades of the building; 14 dials of 18 to 15-ft. diameter in the public rooms; 26 in the various offices, buildings and cabins, and one master clock in the train dispatcher's office.

PNEUMATIC TUBES.—For the prompt dispatch of messages and small packages between the different buildings and offices, a pneumatic tube carrying system has been installed. Each run is of a single tube having terminals at the ends to serve for either dispatching or receiving the carrier. The terminals are normally open and when the carrier is inserted the door at that end is closed and air admitted automatically. The door remains closed until the carrier reaches the opposite end, where it opens an electric circuit and releases the door at the sending end. The lengths of the tube runs vary from 50 to 100 ft., the total aggregating 7,000 ft. The carriers are of leather and of a size to hold messages, baggage checks, packages of tickets, etc.

Lighting.

Small lamps were adopted, not only because of good architectural effect and agreeable quality, but because of effective diffusion. In general, the treatment for different spaces is as follows: The street lighting is by lanterns of moderate power, set on posts 16 ft. high, 45 ft. apart around the building. The arcade is lighted by side brackets containing clusters of Nernst lamps. The public rooms, except the large main waiting-room, are lighted by ceiling chandeliers consisting of rings of Nernst lamps. The main waiting-room because of its great height is lighted near the floor by two rows of cluster lamps on posts with a limited number of side brackets. The ceiling of this room is intended to be left in semi-obscurity, increasing the effect of height at night. The concourse, having its roof and floor largely of glass and steel, was a difficult space to light agreeably and effectively. After various experiments, it was decided that rings of Nernst lamps around columns and ring chandeliers for central spaces were best. The offices are equipped with ceiling lamps for general illumination with local desk circuits provided. The train platforms are lighted by Nernst lamps 20 ft. apart and as high above the floor as possible.

Further data of the lighting arrangement are as follows:

Total number of lighting fixtures in station	21,000
Total equivalent combined candle-power of lights in station	335,000
Feet of conduit used	310,000
Feet of wire (No. 14 to 600,000 cir. mils.)	690,000
Candle-power (mean hemispherical) per square foot, main waiting-room	0.45
Other public rooms	0.60
Offices (general lighting)	1.25
Main concourse	0.35
Platforms	0.25

Electric current for the station is distributed at the required voltages from a main board in the service plant through cables to sub-boards located at division centers of the station. From these sub-boards it is further distributed to local boards in the smaller sections, or in individual rooms, and finally to the room switches or to large groups of lights. All the larger public rooms are supplied through duplicate sets of feeders, one set of which is connected to an emergency bus in the service plant. The main feeders are carried through the pipe subways and in special pipe or wire shafts in the station walls. Branch circuits are generally carried on top of the concrete floor arches and are built into partitions. Feeders and sub-feeders are three-phase; branch circuits are single-phase.

FIG. 5. SERVICE POWER-PLANT BUILDING, PENNSYLVANIA TERMINAL.

Heating and Ventilation.

This complicated requirement covers the heating of the main station as well as of the numerous buildings in the yard and the supply for cars standing in the yard—over an area of about 28 acres. As the type of heating influenced the design and construction of the building itself, the general scheme had to be determined prior to construction.

Difficulties were introduced by the fact that the building has no basement or cellar proper for installing heating mains and radiating apparatus. The rooms vary in size from the main waiting-room, 110×150 ft. and 150 ft. high, down to the usual dimensions of offices, while the occupancy covers the requirements of a railway station, restaurant and an office building. Many rooms are designed to house large numbers of people, and some have only indirect communication with the outside atmosphere, while some are below the street level.

Therefore, it appeared desirable to use a forced-draft system so that air might be taken from suitable places for ventilation in summer as well as in winter. It appeared that portions of the station where forced heating and ventilation were desirable, comprised at least two-thirds of the total space to be dealt with. The indirect system could be extended to the smaller spaces without introducing prohibitive complication or cost. Therefore, it was decided that the indirect system with draft and suction should be adapted for all spaces, except such as baggage-rooms and small isolated buildings in the yard, where direct radiation was used. Use of water, heated at a central point near the engines by exhaust steam and distributed by pumping through a pipe system to locally placed stacks, appeared to be best. Such a system involved small piping, was free from drainage troubles and was convenient for regulation under varying weather conditions. It was found that the indirect system with hot water could be installed at as low first cost as any other.

Steam from the boilers in the service plant is passed through various engines, as described later, and the exhaust is taken into tubular water heaters. Live steam may be used in the heaters when necessary. The circulating pumps are motor-driven centrifugal types. The water returns from the circulating system to be used again. Fresh air is delivered from hoods on the roof of the station building to nine heating stacks and is forced through ducts in the various rooms.

In designing this system it was necessary to provide differential pressures in various rooms, like toilets, kitchens and serving-rooms, to prevent odors from being communicated from one room to another. To economize the heat required in the large public rooms in very cold weather, the discharge from the ventilating fans may be by-passed in part or entirely to the heating stacks, so that the warmed air from the rooms may be used over again in any proportion desired.

PIPE GALLERY.—Reference has been made to the difficulties of finding space for pipes and ducts in the building without cellar or basement. The problem was finally solved by constructing in the floor-truss system, under the main waiting-rooms and baggage-rooms, a pipe gallery having an extent of about two acres. This space is T-shaped and of a height varying from 5 to 7 ft. It is intersected by numerous trusses and latticed bracing, but is sufficiently open to permit of installing a large part of the piping needed. From this gallery a number of vertical shafts are made, so that the main runs are everywhere accessible.

Post-Office Building.

Anticipating the growing importance of an uptown central distributing post-office, the U. S. Government purchased from the railroad company a plot of about 400×400 ft. immediately west of Eighth Ave. as a site for a suitable building. The railroad company reserved the right of easement of all spaces below a plane 20 ft. above the yard track level and certain agreements were entered into regarding the character of the building, location of supporting columns and foundations, and admission of light to tracks underneath. The government has planned and is erecting a monumental building, and, while the railroad company is not responsible for the building proper, the necessities of yard construction required that the foundations should be put in prior to the time when the building contracts were to be let, and simultaneously with the great variety of substructure work in the yard. The railroad company therefore, in conference with the government's architects, planned and constructed the foundations.

The building has been designed by Messrs. McKim, Mead & White. In determining the character of the building, due consideration was given to the proximity of the Pennsylvania Station. In order that it be in harmony with that building, the style adopted was Roman; and at the same time, to give a note of peculiar quality associating it directly with the government class, Roman-Corinthian was the style selected, that of the station being Roman-Doric.

On the ground floor the space allotted to the public is in the form of a wide corridor, back of which is the general working space for distribution, handling, etc. On the westerly front of the building is a private driveway connecting 31st and 33d Sts., giving access to mail wagons and other city deliveries.

At present about 40% of the entire amount of mail originating in New York for outside points is handled by the Pennsylvania R. R. On the heaviest days from 200 to 260 tons are carried in from 12,000 to 16,000 pouches. Practically all this, in addition to that from the Long Island R. R., and possibly later that from the New Haven Road, must be provided for at the new station.

The new post-office building is over the railroad yard, but it is obvious that unlimited track space for standing mail cars or unlimited platform space for loading mail into cars cannot be provided. The value of every foot of space in the yard is so great that provision had to be

made to handle the mail in the most expeditious manner and on very limited track room. Six tracks adjoining four central platforms have been assigned especially for standing mail cars, giving a total maximum storage capacity of 26 cars. It was thought essential by the joint committees of postal and railway officials to develop a complete system of mail handling machinery designed to reach not only the post-office, but through the trucking subways under the tracks in part of the main station building.

Incoming mail in less than carloads lots or in combination passenger and postal cars will be unloaded as the train stands in the station onto trucks and conveyed to the post-office along the platforms or by using the nearest baggage lift, descending to the trucking subway under the tracks and thence to the post-office lifts.

For carload lots and bulk mail, the cars will be switched to the tracks adjoining platform No. 4. Here the mail will be unloaded manually into hoppers on the platform. From here the pouches will be pushed automatically by compressed air rams onto a belt conveyor under the platforms, which will carry the bags to a point where a tilting tray operates to transfer them automatically to a vertical bucket lift, raising them into the post-office mezzanine floor, from which they will be delivered down through spiral chutes to the receiving-mail platform on the first floor or to the basement of the postal building. The compressed-air rams will deliver the pouches to the belt at the proper intervals to be taken by the lift buckets when unloaded there. Co-ordination between movements of the rams, loading trays, conveyors and lifts is further secured by centering the drive in a single electric motor.

The outgoing mail, in less than carload lots, will be handled directly into cars from automobile trucks, which will pass through the trucking subway and baggage lifts along the platforms.

Bulk and carload mail will be delivered into the cars automatically from the post-office floors as follows: The four central platforms adjoining the six car storage tracks are provided with overhead horizontal belt conveyors placed in housings attached to the framework of the building, or in special structures carried by columns set in the platforms. The conveyor belts are driven at a speed of 100 ft. per min. by electric motors. Self-propelled carriages operating on tracks in the housings may be run automatically to any desired position opposite the mail-car doors to trip the pouch into a spout attached to the carriage and thence to the car door. Mail is delivered to the belt conveyors through spiral chutes.

There are four hydraulic lifts, in addition to the conveyor system, affording trucking communication between the buildings, platforms and subways. All the foregoing mail-handling facilities were provided by and will be operated by the railroad company. The special conveyor machinery was designed by Messrs. Marks and Woodwell for the architects of the post-office, and was installed by the Lamson Belt Conveying Co. The work was administered and coordinated as usual through the office of the Chief Engineer of the railroad terminal company.

Express Building.

The terminal operation does not include facilities for the handling of express matter at present, this, for the time, being cared for at the Jersey City and Long Island City stations as before. However, a location has been set aside at the west side of Ninth Ave. admitting of a building 130×180 ft. The yard space under it will provide for one wide loading platform with a track at each side accommodating a total of twelve express cars. There will be two additional platforms in the yard between Eighth and Ninth Aves., accommodating 24 cars. The express platforms will be reached from the building by hydraulic lifts communicating with the main platform under it and with the trucking subway system under the tracks.

Service Power Plant.

This building has been frequently mentioned, and it has been evident that there are numerous and important uses of the power in various forms about the station: (1) heating and lighting; (2) steam, compressed air and water-supply for cars; (3) air supply for signals, tunnel drainage and sewage ejectors; (4) water-supply for buildings and fire protection; (5) hydraulic power for lifts; (6) refrigeration for kitchens, restaurants and drinking water; (7) electric power for buildings, tunnels and yards, for storage battery charging and minor uses, and (8) traction power for trains.] The proper housing of the various facilities for furnishing this power required a large amount of space. Their installation in the station building was out of the question and no central space for a special building was available. A convenient location was available on relatively cheap property on the south side of 31st St., about midway between Seventh and Eighth Aves., directly accessible under 31st St. from the station and yard. The building as erected has a frontage of 160 ft., a depth of 95 ft. and a height of 86 ft. above the curb, together with 49 ft. below. The exterior is of a character to harmonize with the station building.

The general plan divides the building, by a fire wall, vertically into two main parts, the west half being devoted to station-

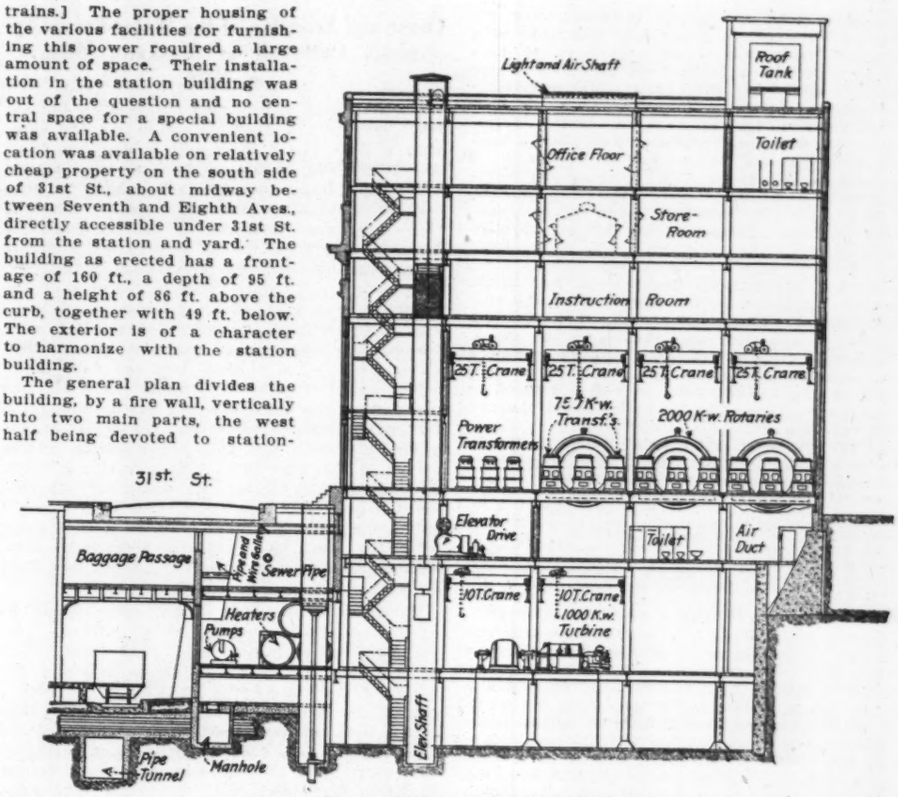


FIG. 6. CROSS SECTION OF SERVICE PLANT.

service equipment and the east to the traction substation, offices and store-rooms.

The basement at the level of the subways below tracks contains machinery foundations, engine piping, garbage destructor, cable splicing chambers and the main entrance to the pipe and trucking subways under the yard. The engine-room is on the floor above, and contains the lighting generators, air compressors, hydraulic power pumps, refrigerating engines, heaters and circulating pumps. The third floor contains the fire and boiler pumps, water storage tanks and a pipe gallery to the station building. The fourth, or street, floor contains the boiler-room, above which are floors for coal storage, and for the fan and economizer rooms. A brick stack terminates above the roof.

The eastern half of the building containing the traction substation has three floors below the street for cable inlets, bus-bar structures and switching apparatus and a main street floor for the rotary converters and switchboards. Two store-room floors are above this, and the top floor is fitted up as offices for the operating staff.

BOILERS.—Space has been provided for ten 525-HP. water-tube boilers (only five of which have been installed on the east side of the room). This equipment is sufficient to provide for the heating at present in the coldest day of winter. When the proposed express and other buildings are erected, additional boilers will be installed as required. The boilers are arranged with shaking grates for the hand firing of small sizes of anthracite. The short stack is supplemented with forced draft under the grates. If tall buildings at some future time adjoin the power plant, the supports will admit of carrying the stack 150 ft. above the roof.

Two economizers are provided and located above the boiler-room. Coal is delivered to the station track in front of the service plant, is vated by a belt conveyor into feeding hoppers in the basement. From here it is hoisted in skips and discharged to the coal bunkers above the boilers by belts. The bunkers have a storage capacity for about a week's supply.

The ashes are dumped into concrete-lined hoppers discharging into hopper cars, on tracks which lead to a storage bunker under the 31st St. sidewalk. From here the ashes are carried by a belt-driven conveyor to the southernmost track of the terminal.

WATER-SUPPLY.—Water for all purposes about the terminal and yard is obtained through two private 12-in. mains connected to independent city mains. The water system provides for boiler feed, fire protection, kitchens, etc., for cooling air-compressor jackets, ammonia condensers, etc., and for general yard use, like car cleaning. The water is metered at the service plant and distributed. An emergency storage supply of 75,000 gals. has been arranged.

For house service there are four steam-driven duplex pumps having a capacity of 300 gals. per min. Two of these obtain their supply from the cooling water of the refrigerating plant and air compressors and are used for flushing water in the several station toilets. The other two are used for pumping fresh water to a storage tank on the roof for various purposes in the station, where the cooling water used for flushing would not be suitable.

For operating hydraulic elevators and lifts there have been provided one 1,500-gal. high-duty compound direct-acting duplex and one 500-gal. compound duplex direct-acting. During the winter, when exhaust steam is needed for heating, the direct-acting pump will generally be used, the high-duty pump being for times of light demand for exhaust steam. These pumps furnish water at 300 lbs. gage pressure.

The drinking-water is taken from city mains and passed through a 400-gal. (per hour) filtering plant in connection with the refrigerating plant. The filtering is done alternately in two cast-iron porcelain-lined drums, first through sand and then through charcoal. The cooled and filtered water is pumped through a special pipe system by two motor-driven centrifugal pumps.

AIR COMPRESSORS.—For operating switches and signals, brake testing, tunnel pumping, sewage ejectors and air cleaners, a compressor capacity of 1,900 cu. ft. of free air per minute is required. As exhaust steam is required during a large part of the year for the station heating, it was found more economical to install steam than motor-driven compressors. Two cross-compound, two-stage, Nordberg-Corliss-valve types have been provided, each with a capacity of 2,000 cu. ft. of free air per min. and compressing to 90 lbs. For the higher pressure required for testing air-brakes, there is a separate plant consisting of two two-stage, tandem-cylinder machines, each having a capacity of 100 cu. ft. and driven through link belts by 30-HP. induction motors.

REFRIGERATING PLANT.—The maximum requirements for the cold boxes in the kitchen and dining-room departments and for the drinking fountains in various rooms and corridors are equivalent to 56 tons of ice per day or an aver-

age of 40 tons throughout the year. For the general refrigeration a complete plant has been installed consisting of two units of 40 tons capacity each, one being kept as spare. These circulate cool brine through a piping system to the station building. To utilize available floor space, vertical compressors were adopted, direct-driven by horizontal engines. The limited supply of ice required for cars and the restaurant is brought in cars through the tunnels and delivered to an ice storage-room in the service plant.

GARBAGE DESTRUCTOR.—The prompt disposal of about five tons of wet kitchen garbage and five tons of dry garbage had to be arranged for. A destructor of the Morse-Boulger type, consisting of furnaces lined with fire brick, has been installed in the basement of the service building. The gases pass from the furnace to the main boilers and thence to the stack. Garbage is brought from the kitchen in closed wagons to a cold storage room, where it remains until time for incineration. It is then dumped on a tray, sorted for valuable articles and charged into the furnace.

LIGHTING AND AUXILIARY POWER GENERATORS.—The approximate requirements for all purposes except traction are as follows:

Station lighting	600	KW.
Tunnel lighting	125	"
Motors for ventilation and station elevators	1,325	"
Motors for sump pumping	800	"
Motors for tunnel ventilation	350	"
Motors for Post-Office mail-handling machinery	55	"
Power for signal system	175	"

Several considerations were guiding in designing the miscellaneous power system. (1) It was determined to use small units for lighting the station building, and for reasons of economy Nernst lamps were adopted requiring 240 volts. (2) The tunnels required a lighting system independent of traction power. (3) Alterating current was needed for operating the signal system. (4) Alternating current could be used also for various motors except in the case of electric elevators. It was decided to operate all motors, except for elevators from the same alternating-current machines, which were used for other miscellaneous-power supply. For simplicity, it was decided to operate the elevators from the traction circuits at 650 volt. (5) For the minor storage-battery, telephone, and clock services, low-voltage direct current was needed and small motor generators were used, converting the alternating current into direct.

The following list gives the variety of electric currents required. Of these, only the 650-volt direct current is not produced by the service plant auxiliary machines.

11,000-volt,	A. C.,	60-cycle,	High-tension service feeders
2,200 "	"	"	Signal lines
220 "	"	"	Signal lines in yard cabins
650 "	D. C.		Third-rail feeders and office elevators.
420 "	A. C.,	60-cycle,	Motors
240 "	"	"	Lights
220 "	D. C.		Elevator control
110 "	"		Car-battery charging and exciters
35 "	"		Baggage-truck battery charging.

Two 1,000-KW. steam-turbine generators supply through step-up transformers three-phase alternating-current at 11,000 volts and 60 cycles. They operate non-condensing, the exhaust steam being passed through the heaters for the station building. For seasons when the exhaust cannot be so utilized, these machines are shut down, and the power supply is taken from Long Island City.

(Concluded in next issue.)

Tests of Steel Rails containing 0.15% of metallic titanium have been made on a 9° curve on the Baltimore 6 Ohio R. R. The curve is on the eastbound track, with a down grade of 0.9%, and the track is rock ballasted. The rails on the curve were laid Aug. 9, 1910, and measurements of the cross-section of the rails were made on July 12, 1911, after the rails had been in service 338 days.

Besides the bessemer steel rails treated with titanium, a number of open-hearth rails were laid on the curve. These rails were higher in carbon than the titanium bessemer rails, the average of the titanium rails being 0.47 carbon and of the open-hearth, 0.64%. The average wear of the open-hearth rails was 0.25 and of the rails with 0.15% titanium the wear averaged 0.147%.

A full report of the tests has been published by the Titanium Alloy Mfg. Co., Oiver Bldg., Pittsburgh, Pa.

Cause and Sequelae of the Water-Borne Typhoid Outbreak at Mankato, Minn., in 1908.

A remarkable water-borne outbreak of typhoid fever occurred in Mankato, Minn., in the summer of 1908. In a short period some 511 cases of typhoid and 35 deaths were reported among some 10,000 water takers. As frequently happens certain types of water-borne typhoid outbreaks, this one was preceded by an epidemic of diarrhea, affecting within a very brief period something like half the population of the city. One of the particularly notable features of the outbreak was that the water-supply of Mankato is taken from relatively deep artesian wells which had never been suspected of pollution. Later events seem to show that the wells themselves were not at fault, but that the water was polluted by the backing up of sewage along the line of the main connecting the wells of the pumping station and into the manhole, giving access to the head of one or more of the wells. Another notable feature or result of the outbreak was a large number of threatened lawsuits to recover damages on account of typhoid cases or deaths. Only two suits were actually brought, the claims of the other parties having been filed too late.

In our issue of Feb. 11, 1909, we published an illustrated article on the Mankato outbreak, prepared for this journal by Prof. F. H. Bass, Assoc. M. Am. Soc. C. E., then Engineer, and now Director of the Engineering Division of the Minnesota State Board of Health. At the time that article was written, Prof. Bass was unable to give many details regarding the exact anture of the pollution of the water-supply, owing to the litigation just mentioned. The suits have since been settled out of court, following a decision on demurrer to the effect that the complainants had a standing in the court. This decision is given at the close of the article. We are also enabled to give an account of the way the water is now believed to have been polluted and the measures taken to guard against a similar occurrence in the future. This account is based on an advance copy of portions of the forthcoming biennial report of the Minnesota State Board of Health, supplied to us through the courtesy of the Board and the kindness of Prof. Bass. We have also had the benefit of assistance in the preparation of this article from Dr. H. W. Hill, Director of the Epidemiological Division of the Board.

We are informed that since the material for the main portion of this article was placed in our hands the State Board of Health has arranged for the publication of an official monograph on the epidemic, which will first be issued by "The Journal of Infectious Diseases" (Chicago, Ill.), probably in the number for November, 1911, but may subsequently be obtained from the State Board of Health of Minnesota.

As we have already stated, the water-supply of Mankato is taken from artesian wells. These wells are located not far from the Minnesota River and a number of them beneath Washington Ave., with their heads in close proximity to an outfall sewer. The location of the four wells beneath the street, as also the location of the sewers and certain water mains, is shown by Fig. 1, while Fig. 2 shows cross-sections of the four wells. These sections, it appears, have been made up, largely or wholly, from such information as could be obtained since the typhoid outbreak, combined with actual records of changes which have been carried out in the way of repairs to the wells to prevent further pollution.

Before saying more about the wells, it should be noted that on June 25, 1908, the day of the beginning of the epidemic of diarrhea, which was the forerunner of the typhoid outbreak, there occurred the culmination of the highest flood in the Minnesota River since 1881. An extremely heavy rainfall in Mankato occurred on the same day. A number of streets, including Washington Ave., were flooded to a depth of over 1 ft. It was this flood which, it appears, was responsible for backing up the sewage so that it polluted the water-supply and caused the typhoid outbreak.

Descriptions of the several wells may be quoted from the report, already mentioned, here also Figs. 1 and 2):

The McCarthy well is of recent construction and probably did not contribute to the disaster. The Old Well was sunk some twenty years ago and both the overflow to the sewer and the suction pipe were badly corroded. Holes of 1½ ins. in diameter entirely through the pipe were found in both. I saw one of these plugged with a poorly fitting piece of wood. The sewage from the sewer immediately beneath this well pit certainly backed up into the pit and as surely was drawn through the holes in the suction pipe to the pumps and distributed.

The Front St. well was of recent construction and in good condition, no contamination from it was possible.

The Broad St. well was found in a condition of extreme neglect. When first found by the writer, this well pit was filled with water to a point within 4 ft. of the surface of the ground, and the lower outlet to the sewer was dimly visible through the still standing water. It was obvious from the debris in the crevices between the brick side walls and plank covering that the water in the pit had been as high as the surface of the ground.

A curiously placed pipe in an inclined position which appeared to be a branch of the well casing occupied the center of the pit. No one in Mankato could tell what this pipe was, although many guesses were hazarded; later a workman stepped upon it and it rolled to one side. This incident is mentioned to bring out the remarkable fact that previous to the epidemic absolutely no one was familiar with the construction of this well. Later upon the subsidence and clarification of the water in this pit, it was found that the supposed top of the well casing was in reality but one end of the vertical arm of a T, the other end terminating a foot or more above the bottom of the pit. It is absolutely certain that the sewage backed up into the pit. Nearby manholes show that the sewage backed up to within 18 ins. of the surface of the street or about 6 ft. above the bottom of the pit. Thus sewage was pumped directly into the water mains of the city.

At the outlet of the sewer passing the Broad St. well was a large manhole or pump-pit, divided into two parts by a wall at right angles to the line of the sewer. In the upstream division of this pit was a small centrifugal pump which at the time of high water pumped sewage over the top of the wall. At ordinary stages of water in the river, the sewage flowed directly through a gate in the wall. At the time of this flood, the pump was unable to handle the large quantity of sewage coming to it, hence the backing up into the system of pipes in the city.

The engineer at the pumping station was notified of the appearance of sewage in the engine rooms of a nearby flour mill by attendant at the latter place. The gate in the dividing wall of the well was opened and the danger was supposed to be over. However, the unusual appearance of the city water-supply was soon noticed and the President of the Board of Public Works was notified by the health officer of the city. He refused to stop the pumps as there was not sufficient water in the reservoir for emergency in case of fire.

When the pumps were not running, it was customary to allow the overflow of the Broad St. well to run through the suction pipe, which sloped downward toward the pumps, through the pumping station to the river. There is some uncertainty as to the exact time when the gate in this pipe was opened, but during a certain period the sewage was high enough on the lower side of the gate to allow it to flow back into the suction main in considerable volume. There is evidence to show that this must have occurred, since the sewage backed up to the boiler-room floor and its route thence was through a manhole into which the above mentioned suction pipe discharged.

We are informed by Dr. Hill that Mr. Wilson, who became city engineer after the epidemic, unearthed "an extraordinary maze of pipe connections which permitted the contamination of the wells near the pumping station."

Before proceeding to describe the repairs made to avert possible future typhoid outbreaks, the report from which we have just quoted makes the following pertinent remarks on the dangerous state of ignorance existing in Mankato prior to the typhoid outbreak in regard to the dangers of the polluted water-supply, and urges the importance of state supervision of the water-supply of Mankato:

All of these conditions making possible the contamination of the water-supply were unknown to the City Council, the Board of Public Works, to the Health Commissioner or any one else.

Mankato citizens prided themselves on their supply of pure water, but their assurance was founded on mere assumption and a terrible calamity resulted.

No one in particular was to blame; the form of government which divided the responsibility and the lack of appreciation of the need of care in obtaining and maintaining proper sanitary conditions was the underlying cause.

Such conditions now probably exist in more than one Minnesota city. The Engineering Division of the State Board of Health should be in a position to examine regularly all sources of water-supply. Such inspection is at least of as much importance to life and property as the inspection of steam boilers which are regularly inspected.

Reconstruction and Repairs.

In making changes in the water-supply since the typhoid outbreak, measures have been carried into effect to increase the capacity of the supply, as well as to render pollution impossible in future. The first thing done was to rebuild the Old Well. This well originally had a 10-in. casing, carried to rock at a depth of 130 ft. The well was reamed out at a diameter of 10 ins.,

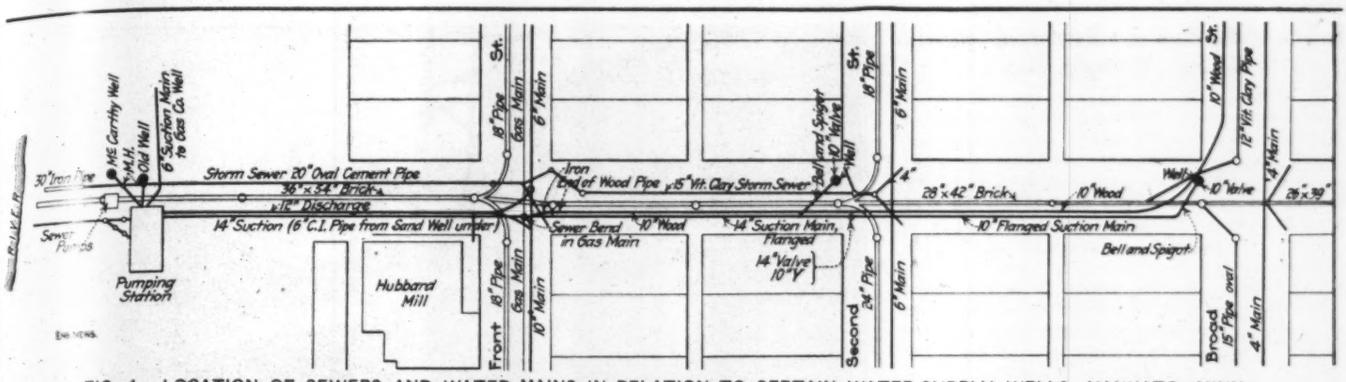


FIG. 1. LOCATION OF SEWERS AND WATER MAINS IN RELATION TO CERTAIN WATER-SUPPLY WELLS, MANKATO, MINN.

(The map was drawn in March, 1910, and shows some changes made since the typhoid outbreak of 1908. A more extensive map appeared in Eng. News, Feb. 11, 1909.

and an 8-in. casing was placed inside the old 10-in. casing, with a rubber packing, so as to make a tight joint at the rock line. The second hole shown alongside this well in the section, Fig. 2, indicates an uncompleted well which was found when repairs were being made. A pipe was carried up from the top of this well to a point above maximum high water in the river and then extended to the river to provide for "any occasional overflow" from the well.

The connection between the Second St. well and the suction pipe was lowered to give an increased flow, and a watertight manhole was built here.

The suction main on the McCarthy well was lowered, and cast-iron was substituted for wrought-iron pipe at this point.

The most radical changes were made at the Broad St. well, which was reconstructed. On this work we quote again from the report, already mentioned, as follows:

The old bore hole could not be exactly located because the well casing had long since been corroded. A length of old smoke stack had been put in around this casing and evidently packed with flax seed. Also a length of cast-iron pipe which had evidently been used at an attempt for repairs was found. After making numerous attempts to locate the well in the rock 90 ft. below the surface, a new well was put down. A 20-in. pipe was put down to a depth of 4 ft. into the rock and a 10-in. pipe was put inside of this. The space between these two pipes during construction was filled with fine sand. Eighty-six cartloads were used in all. The water during the sinking of this well was coming up on the outside and the purpose of putting the sand inside was to allow it to pass down around the lower end of the pipe and be carried up by the water to stop the flow on the outside. This expedient succeeded; after a while the water came up inside the 10-in. pipe which was then driven to a depth of 4 ft. below the 20-in. pipe. Finally, the space between the two pipes was filled with cement mortar, making a cement pipe which would always be in place after the destruction by corrosion of the iron pipes. A concrete manhole was constructed around this well. The bottom was left open and covered with sand, so that if the old well should by any chance break out it would come up through the manhole and make itself evident.

The conditions of the suction main and changes in it found necessary and carried out are described in the report as follows:

The second operation was to determine the exact location and condition of suction mains. These were found to be overflows or drains from the wells to the pump rather than true suction mains; the pumps would not operate until there was about three pounds pressure from the wells, hence the full capacity of the wells could not be realized.

All pipe near the pumping station was found to be of wrought iron and in bad condition. There were many holes extending through the pipe and several joints entirely rusted through; the main from Broad St. to the pumping station had many leaks. This old main was torn up and relaid on a grade of 0.10% upwards towards the pumping station. Flanged pipe of 12 ins. in diameter from the pumping station to Front St., 10 ins. in diameter from Front St. to Broad St. was used. The old cast-iron pipe from the pumping station to Front St. was in good condition, but from Front St. to Broad St. there were found 28 breaks; in two places the pipe had broken off entirely. In one break it had been attempted to mend the pipe by wrapping canvas around it. A cold chisel could be driven through the pipe at one blow.

Changes in the sewage system of the city in the vicinity of the pumping station involved first the cutting off of a connection between the main outfall sewer, shown in Fig. 1, and a drain from the pumping station. In addition, a new sewage pumping station was built at this point. In it there were installed an 8-in. centrifugal pump, having a capacity of 10,000 gals. per min., geared to a 65-HP. motor, and an 8-in. pump, of 1,000 gals. capacity per min., geared to a 15-HP. motor. The reconstruction work described was carried out under the direction of Mr. John Wilson, City Engineer of Mankato.

Court Decision as to Liability of the City for Personal Damages Due to the Typhoid Outbreak.

As stated at the beginning of this article, two suits were brought against the city, following the epidemic. These suits were brought by the widows of Louis Eugene Keever and F. R. Flanagan for the recovery of $5,000 each for the deaths of their husbands, said to have been caused by the pollution of the city water-supply. In trial court the suit was thrown out on a demurrer, but this decision was reversed by the State Supreme Court in a decision handed down by Justice Jaggard. We quote his syllabus of the decision as follows:

Blue Earth County—Delia Keever, as administrator, appellant, vs. the City of Mankato, respondent, Kate Flanagan, as administrator, appellant, vs. the City of Mankato, respondent.

A complaint charged that defendant city negligently allowed the supply in its water-works system to become polluted with poisonous substances and large quantities of filth and sewage to escape into and saturate its water-supply, by reason whereof plaintiff's intestate contracted typhoid fever and died as a consequence.

On demurrer it is held: (1) The municipality was liable for its negligence in its private or corporate capacity, and was not exempt because it was carrying out a governmental function.

(2) Under Sec. 4403 an administrator of a person whose death was due to the wrongful act of a municipality may maintain an action for damages consequent thereon. Malone vs. City, 40 Minn., 406; Orth vs. Village, 57 Minn., 207, followed.—Reversed.

Opinion by Justice Jaggard.

As already stated, after this decision the two lawsuits were settled out of court.

Automobiles Exported from the United States during the first seven months of 1911 amounted in value to more than $9,000,000. During the same period, $1,500,000 worth of tires were exported and $2,000,000 worth of automobile parts, making the aggregate value for the seven months about $12,500,000. The total value of automobiles and parts exported in 1901 was less than $1,000,000.

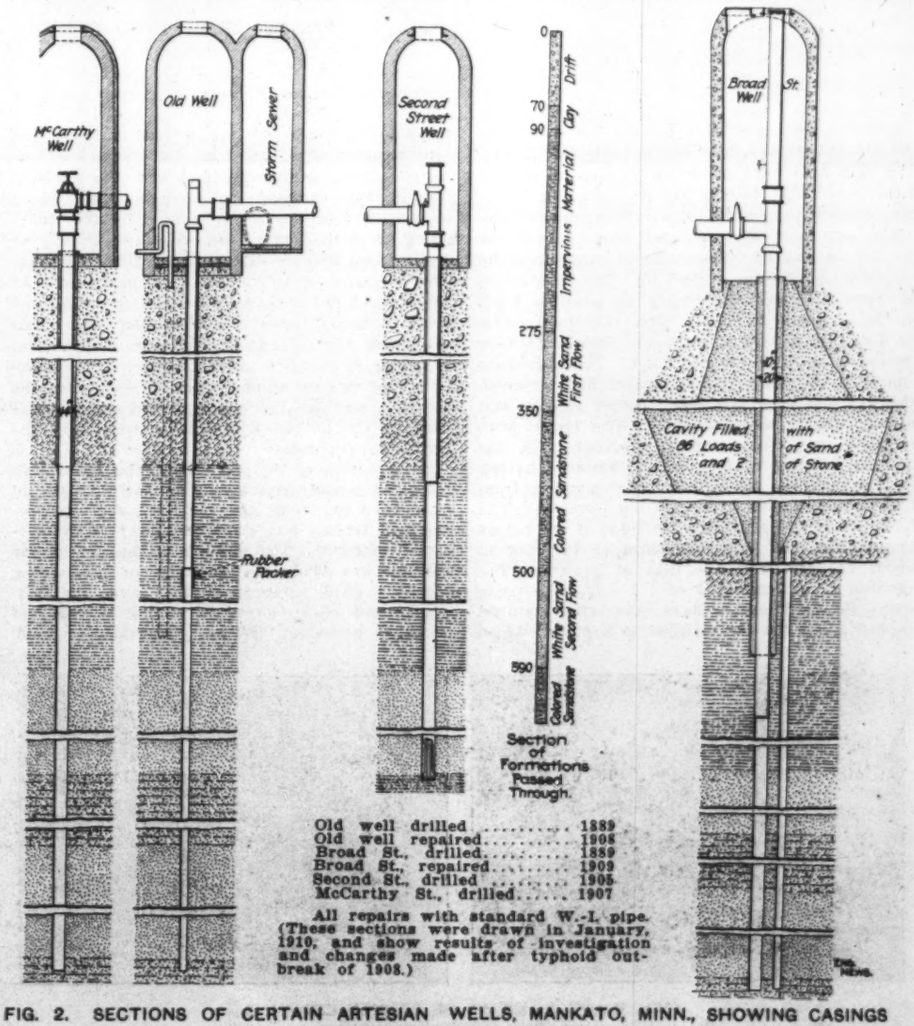


FIG. 2. SECTIONS OF CERTAIN ARTESIAN WELLS, MANKATO, MINN., SHOWING CASINGS AS REPORTED AND AS REPAIRED.

Failure of the Roof of a Reinforced-Concrete Building, Winnipeg, Man.

On Sept. 7, 1911, nine bays of the roof of the reinforced-concrete warehouse of the Canadian Fairbanks-Morse Co., under construction, at Winnipeg, Man., collapsed while props were being removed and two workmen were killed by the falling material. The floor below the roof held firm under the impact, and all structural damage was confined to the section of the roof that fell.

The building is four stories in height above street level and is of reinforced-concrete skeleton throughout, with floors and roof of similar materials. The main floor is designed to carry 250 lbs. per sq. ft. live load, the three floors above each to carry 200 lbs. per sq. ft. and the roof to carry 40 lbs. per sq. ft. The reinforcing used was manufactured by the Trussed Concrete Steel Co., and excepting in the roof columns, was of deformed steel; in the beams and girders the Kahn trussed bars and "cup" bars and in slabs the "cup" bars alone were used. The beams were built as continuous girders. Each panel of the roof that failed is 23 ft. long and 19 ft. wide, c. to c. of columns, and divided into two 9½ × 23-ft. bays by a longitudinal beam, spanning between the short girders. The size and reinforcement of the members are as follows:

FIG. 1. ROOF OF CONCRETE BUILDING COLLAPSED SEPT. 7, 1911, WINNIPEG, MAN.

Member.	Section. ins.	Reinforcement.
Long girder	14 × 18½*	3—¾ × 2-in. trussed bars. 2—⅝-in. cup bars over ⅔ length.
Short girder	12 × 18½*	Inverted over columns; 2—¾ × 2-in. trussed bars over ½ length.
Intermediate girder	12 × 14½*	2—¾ × 2-in. trussed bars. 1—¾-in. cup bar over ⅔ length. Inverted over girders; 2—¾ × 2-in. trussed bars over ½ length.
Column	12 × 12	4—½-in. round rods at corners, wrapped every 12 ins. with ½-in. wire.
Slab	3½	⅜-in. cup bars, 10-in. c. to c. short way and 24-in. c. to c. long way.

*Exclusive of 3½-in. floor slab.

Into the bottom of each column the rods from the column of the floor below projected 12 ins. The slab rods were permitted to stop at the intermediate beam, thus allowing only a 4-in. bearing, without lap, of the slab reinforcement here.

The concrete of the roof was poured on August 18 and 19. The columns supporting the roof were poured on August 15. The proportions of aggregate were as nearly as possible 1:2:3 in the columns, about 1¾ bbls. of cement being used per cubic yard of concrete. The floors were poured from a mixture of 1:2:4. The aggregate consisted of water-washed gravel not screened, the particles of gravel being rather smooth and round, and of screened sand. The forms were stripped from the columns on August 25, the general practice throughout the building having been to strip the forms from the columns in from four to eight days after pouring concrete. The forms were stripped from the sides of the beams and girders and from the slabs of the roof in about 11 days from the time of pouring. The shoring, which consisted of 4 × 4-in. pine posts with crossheads and wedges, was being removed on Sept. 7, when the collapse took place. Upon the shoring and under the bottom of the forms two wedges were used and the method of removal of the shoring involved first the slackening of these wedges, the foreman being under explicit instructions to be warned by the tightening up of the wedges once loosened, and to refrain from further disturbance of the shoring. This warning seems to have been neglected. At the time of the collapse, there lay upon the concrete a triple layer of tarred felt and pitch placed on Aug. 21 and 22, and over that a 1-in. boarding on 2 × 4-in. nailing pieces. It is probable that this covering, placed so soon after the concrete was laid, served to retard the free access of air to the floor and consequently the set of the concrete.

Examination of the concrete in the failed slab indicated considerably less of setting than should have been expected, and the process of hardening and setting has since continued in the collapsed material. The concrete at the time of the collapse was dead, that is, it did not have a ring such as good concrete should have, and evidently had been delayed in setting by the cold weather prevalent during its setting period. Throughout the remainder of the building the concrete seems to be quite dense and the forms to have been well filled. There is no evidence of a skimping of cement or of anything but the best practice on the part of the contractor. So far as can be noted, none of the reinforcing steel failed.

FIGS. 2 AND 3. VIEWS OF BROKEN COLUMNS.

Figs. 2 and 3 are views of two of the columns found within the fragments of the collapse. It will be noted that the fractured ends of these columns show the outlines of typical crushing, and that the reinforcing steel which projects from the columns is curled down and inward as would be the case with this kind of failure. It would seem fairly evident that the columns were the first to fail. With the shores removed, each column was called upon to bear about 22,000 lbs. or a trifle over 150 lbs. per sq. in. The evidence is not clear that the shores were altogether released. The evidence of one workman engaged upon the removal of the shores is to the effect that the wedges were not loosened by hammering, but seemed to tighten up again after each blow. A succession of extra heavy blows finally loosened them and the collapse followed.

There is some question about the quality of the cement, which was not tested but taken on the word of the Canada Cement Co., which supplied it. This company, since the merger some time ago, has controlled by far the greater proportion of the output of Portland cement in Canada. As a result of that merger all of the well-known trade names of cement have been done away with and all cement controlled by the parent company, regardless of its mill, is branded on the bag "Canada Cement Co." Under this arrangement it is not always easy to determine from what mill the particular sample of cement has been shipped and except in the case of certain customers, chiefly the larger corporations, it has not been the practice to make a full series of tests prior to using the cement. In the building in question, then, nothing is known of the cement except that it is from one of the mills controlled by the Canadian Cement Co., and as such, is guaranteed as complying with the requirements of the Canadian Society of Civil Engineers.

A coroner's jury rendered a verdict of unavoidable accident, with recommendations that there be an enforced testing of all cement used on structures whose failure might endanger life.

The architects in charge of the building are Messrs. Brown & Vallance, and the contractor George H. Archibald & Co., both of Winnipeg.

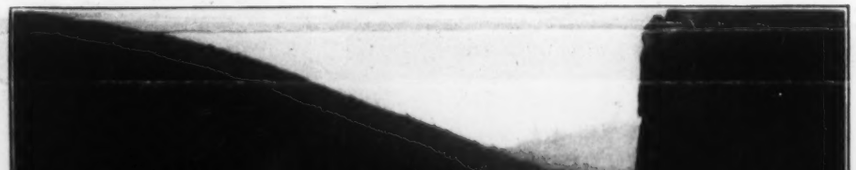
The above account has been prepared from information received from two different Winnipeg engineers, who are familiar with the facts in the case.

FIG. 1. A TYPICAL PILE OF WRECKAGE IN AUSTIN.

FIG. 2. FRAME DWELLINGS WRECKED BY THE FLOOD.

FIG. 3. CHANNEL MADE THROUGH THE VILLAGE BY THE FLOOD.
(Note small size of stream after flood subsided.)

FIG. 4. GENERAL VIEW OF WRECKED DAM LOOKING UPSTREAM.



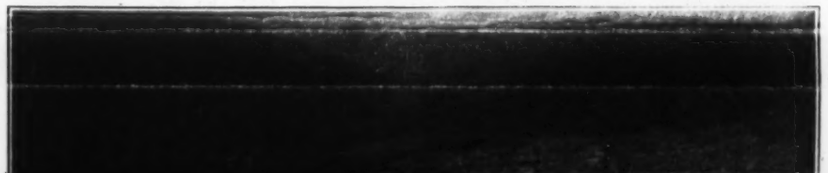


FIG. 5. VIEW OF UPSTREAM FACE OF DAM.
(Section at right is undisturbed portion at west end.)

FIG. 6. VIEW ALONG DAM FROM EAST END; EAST GAP IN FOREGROUND.

FIG. 7. LOOKING UPSTREAM THROUGH BREAK WEST OF SPILLWAY.
(Note extensive shattering at toe of undisturbed part of dam.)

FIG. 8. VIEW DOWNSTREAM THROUGH EAST GAP.

PHOTOGRAPHS AT THE SITE OF THE DAM FAILURE AT AUSTIN, PA., SEPT. 30, 1911.

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UNIV. OF MICH.

ENGINEERING NEWS

A Journal of Civil, Mechanical, Mining and Electrical Engineering

Published every Thursday by

THE ENGINEERING NEWS PUBLISHING CO.

290 BROADWAY, NEW YORK

BRANCH OFFICES

CHICAGO: 1138 Monadnock Block
WASHINGTON, D. C.: Home Life Building
SAN FRANCISCO: Monadnock Building
RUDOLPH MOSSE, BERLIN AND HAMBURG, GERMANY
S. P. MARUYA & Co., Ltd., TOKYO, JAPAN

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Selecting the Manager of a Municipal Water-Works by Popular Vote.

An illuminating illustration of how *not* to carry on the business of city government is furnished by the following advertisement in a recent issue of the Atlanta "Constitution":

To the Voters of Atlanta:

I announce myself as a candidate for general manager of the Atlanta water-works at the ensuing city election and subject to a primary if one is called.

Very respectfully,
Park Woodward.

It happens in the present case that Mr. Woodward has been for many years in charge of the Atlanta water-works and his reelection is doubtless taken as a matter of course. The business interests of the city would doubtless make themselves heard in no uncertain tones if an incompetent politician were to seek election as Superintendent of the municipal water-works.

But the fact that good results may be secured under a bad system is no defense of the system. Can the thousands of individual voters in Atlanta, either in the election or in the primary exercise any intelligent choice in picking out a man to have charge of the city's water-supply? To state the question is to answer it. Whatever the defects of the Commission plan of city government may be, it does away at least with such absurdities as the selection by popular vote of the men whose work demands engineering and scientific ability.

How long would any manufacturing corporation escape bankruptcy if its shop superintendent and chief draftsman and sales manager and head of the publicity department were elected by popular vote of some hundreds or thousands of stockholders?

The United States is the only leading nation of the world where such unbusinesslike methods of carrying on city governments still exist. It is also the only leading nation in which graft and inefficiency in municipal administration has become a national disgrace.

The City of Atlanta voted last month upon the acceptance of a new city charter. In the review of this charter appearing in the same issue of the "Constitution" with the above advertisement, a great deal is said about the checks and safeguards which have been provided to prevent misuse of the city's funds.

Charter drafters in the legal profession generally devote great pains to the invention of systems to prevent the wrong things being done. It is just as necessary, however, if efficient and economical city government, is to be secured, to enable the right things to be done and done promptly. Most of all is it necessary to provide systems which will enable competent men to be placed in charge of technical matters and kept in charge through successive years without regard to politics.

Power-Apparatus Ratings.

There are evidences of a growing dissatisfaction with the zeal of salesmen for power-plant machinery which leads them to give such apparatus as steam engines and turbines, water wheels, electric generators, pumps, etc., capacity ratings which closely approach the ultimate output, or to guarantee high efficiencies which can be proved only by careful test. The salesman takes a gambler's chance that such tests will never be made. Use of such ratings gives an impression of low unit investment, which attracts the purchasers to one type of apparatus where perhaps another might be better. Some day the purchasers wake up and find that their plant has little overload capacity for excessive peaks or emergencies, or perhaps they learn from the operating cost sheets that the alleged high efficiency apparatus has not earned dividends. Judging from Mr. Dreyfus' recent paper, extracts from which appear elsewhere in this issue, this practice of making extravagant guarantees of efficiency is particularly prevalent in the steam-turbine field.

The burden of effecting a reform rests on both the sinners and those sinned against. On the one hand, manufacturers will have to forbid the use of ambiguous capacity ratings or the making by salesmen of excessive guarantees for efficiency. They must conscientiously resolve to claim only what they can make good under impartial test. On the other hand, the prospective purchaser ought to be wary of accepting guarantees of high efficiency unless they are to be subjected to test. He, or his engineer, should know the results which have been secured under similar conditions elsewhere, and should be guided by such records, in planning their design and purchasing apparatus.

Service Equipment of the Pennsylvania Tunnels and Terminal.

Mention of the Pennsylvania tunnels and terminal in New York City brings to mind various visions of monumental works like the actual boring of the tubes, the excavation of the site or the construction of the impressive station building itself. What has been published in the past was necessarily related to these pieces of primary engineering work. It is only by second thought of the completed project that these things are seen to be only the skeleton of the project and that a multitude of smaller pieces of engineering design and construction have to be executed before any of the work is capable of rendering service. Standing alone, each of these minor works would command attention by the size, by the peculiar requirements of design, or by features of design.

To show, in a measure, what is involved in some of these necessary hidden works, we give considerable space in this and the following issue, to extracts from a very complete paper by Mr. George Gibbs, Chief Engineer of the Pennsylvania Tunnel & Terminal R. R. Co., before the American Society of Civil Engineers. Mr. Gibbs' paper makes a formidable volume, although it is itself only a summary. Any abstract which we could present of such a paper within reasonable limits of space, must necessarily appear inadequate and sketchy to those most familiar with the work itself; but it should give others useful information as to the number of interesting engineering problems which had to be solved.

No one entering the Pennsylvania station, we think, for instance, would appreciate that over a million dollars was invested in work done beneath the level of the tracks. But one has only to glance at the part of Mr. Gibbs' paper discussing the subways to realize how necessary they are to the success of the whole project.

The power expended in the various utilities about the tunnels, station yards and auxiliary buildings required the establishment of a power plant of very considerable capacity. Few passing through the station will realize that to furnish adequate service nearly 3,000 HP. is required, in addition to the supply needed for hauling the trains back and forth. Few think of the care that has been expended upon the heating and lighting, upon the supply of good drinking-water, upon the disposal of refuse, or to provide for the adequate handling of the vast amounts of baggage and mail matter which goes with the operation of a great railroad terminal like this.

Prompt and convenient service depends upon attention to many little details the presence of which the passenger is ordinarily quite unaware. For instance, it needs but a moment's consideration to show how essential it is, in a station of this character, that the trains shall depart on schedule time and without any possible confusion from personal orders or hand signals passed from train to concourse gate, etc. One would almost overlook the dispatching signals which make this promptness possible and still admit passengers to the trains up to the last second of departure without danger to life and limb.

In short it is essential that such a paper as Mr. Gibbs' be carefully read by any one who would take more than a passing look behind the scenes.

A Remarkable Timber Bridge of Up-to-Date Design.

In these days when steel and concrete occupy the attention of structural engineers almost to the exclusion of other materials, it is worth while to call attention to the fact that there are still regions where timber structures are being built to carry heavy loads and on fairly permanent work. Having generally in mind the hatchet-and-saw woodwork ordinarily allowed in temporary construction, such as one is apt to find in falsework and the rougher types of concrete forms, the average engineer does not consider any longer the finer details of wood design, and often leaves the designing as well as the building to the carpenter's foreman in charge of the work. Every structural engineer will be interested, therefore, to note the striking exception to the rule in the Columbia River bridge described by Mr. F. L. Pitman on another page of this issue. A timber bridge comprising nine 150-ft. Howe trusses, one 240-ft. swing draw span and nearly 1,000 ft. of heavy timber trestle, approaches, if it does not exceed the largest wooden bridges of a past time when wood was the chief material available to the American bridge builder. In strength and size of members it far exceeds any of these older bridges.

It is stated that the life of the bridge is estimated at 10 or 12 years. This would seem to be an ultra-conservative estimate. All over the eastern part of this country there are to be found timber truss bridges of the Burr and Howe types, which are even now safely carrying traffic after 50, 60 and even 100 years of constant use. To be sure the traffic is generally light on these old structures, nowhere approaching the heavy trainload requirements of the Columbia River bridge. On the other hand, the modern details of joints and protective coatings in the Columbia River bridge should serve to offset the deteriorating effects of heavy loads. As has been the custom for some years on the Canadian railways, where timber bridges have been widely used on account of the remoteness of the steel market and the nearness of the forests, on the Columbia River bridge great attention was paid to the design of joints and in every point of contact of

timber on timber an iron plate or casting has been introduced to reduce the possible rotting. While the necessities of traffic may require the replacement of this bridge with steel in the next decade, it does not seem probable that the cost of upkeep or possibility of failure should bring about its removal for at least twice that time.

Although it bears no direct relation to the truss structure or the design of the bridge as a whole, the extra heavy cross-bracing of the trestle bents in the approach will bear inspection. It is very rare that stability is made sure by such a close spacing of braces, but the peculiarly severe conditions of the river regime seem to warrant it in this case.

Justifiable Homicide of the Proposed Charter for New York City.

The proposed charter for New York City, on which we commented adversely in our issue of Aug. 17 (p. 210), was smothered in the State Senate on Sept. 30, after having passed the Assembly a few days earlier. The charter as originally proposed violated many principles of efficient municipal government, had no representative public support, but almost unanimous public opposition outside of partisan political circles, and in both origin and means of promotion violated almost every principal of municipal home rule. Eleventh-hour amendments designed to meet some of the many objections to the charter made the document more of a hodge-podge than before. The defeat of the charter is a striking illustration of the force of united public opinion, guided by intelligent leaders, and voiced by the public press. It is to be hoped that the next attempt at a new charter for New York City, which will be the fourth or fifth within a few years, will be placed and left in the hands of competent charter framers instead of being controlled by partisan politicians.

Another Austin Dam Failure and Its Lessons.

The terrible disaster which destroyed the village of Austin, Pa., last Saturday with a loss of two hundred or more lives, is a shock to the engineering profession hardly less appalling than that experienced when the Quebec bridge fell four years ago.

So far as the newspaper reports have informed the world, the structure was designed and built under direction of a competent engineer of good reputation. The structure fails suddenly with great loss of life, and the public jumps to the conclusion that no engineer is reliable, and that no dam is safe.

We wish, first of all, to combat this idea. Most emphatically do we declare that it is within the resources of engineering to so build such a dam as that at Austin that it shall be safe and secure. It is not our province to sift and apportion the responsibility for this sad calamity. It *is* our duty to say that the occurrence is without excuse; and that if the dam had been built with proper precautions and proper regard for safety the failure would never have occurred. There is no mystery in the art of dam construction and no problems in their design which competent engineers are not capable of solving with accuracy and reliability.

But, it may be said, why then do dams and bridges and other engineering structures fail, even when engineers of good reputation are employed in their design? We can answer this question in a word. These disasters occur because too frequently those who build these structures *buy just as little engineering and buy it just as cheaply as they possibly can.*

They do not quite dare to go ahead with an important structure without some sort of engineering advice; but they get along with just as little as possible; and too often they reject so much of it as does not suit their fancy or their purse.

Of course not all engineering work is carried on in this way. If it were, accidents and failures to engineering works would be far more numerous; but we do know that far too much of the work of engineers is done under conditions which prevent them from really bearing the responsibility that is nominally theirs.

It is farthest from our desire to say anything at this time which may needlessly add to the burden of those, whoever they may be, on whom responsibility for this disaster may be placed and who are without doubt already suffering tortures untold. We should be derelict in our duty, however, did we not point out the salient facts in connection with the disaster, not for the purpose of locating the blame, but to indicate how similar calamities may be avoided in the future.

The article which we reprint elsewhere from our issue of March 17, 1910, describes a partial failure of the dam at Austin which took place in January, 1910, a failure which the careful reader will see might easily have caused at that time just such a calamity as has now occurred.

The earlier failure of the Austin dam, by a most remarkable coincidence, occurred from almost the identical cause which destroyed the Colorado River dam at Austin, Tex., on April 7, 1900. The dam at Austin, Tex., was slid forward on its base until it toppled over. The dam at Austin, Pa., was slid forward together with a stratum of the underlying rock upon which it was founded. By great good fortune the movement stopped before the yielding dam section was crushed or overturned and emergency measures were taken to draw off the water from behind the dam and relieve the pressure upon it. No one seems to have realized, however, that in that condition all assurances as to the safety of the dam had wholly disappeared. The pond above the dam was again filled with water and the sole concern of those in charge seems to have been to stop the leaks, utterly oblivious to the fact that this broken structure, its bond with the underlying rock destroyed, was a veritable sword of Damocles hanging over the village below.

It was bad to build a dam on such a foundation, with no greater precautions to prevent water getting under it; but it was ten times worse to attempt to use the structure in its weakened and perilous condition after the partial failure of January, 1910.

Who took the responsibility of deciding to run this risk, the courts will doubtless have to determine.

Mr. T. Chalkley Hatton, of Wilmington, Del., the engineer who made the original designs for the dam informs us that in February, 1910, after the failure above noted, he submitted to the owners of the dam plans and recommendations for its repair and strengthening which he had prepared after consultation with Mr. E. Wegmann, Consulting Engineer, of New York City. Mr. Hatton has no information, however, whether these recommendations were ever carried out or what, if anything, was done to repair the dam after his report in our issue of March 17, 1910.

The plan of repair proposed by Mr. Hatton was, we understand, first to stop the flow underneath the dam by a deep cut-off wall on the upper side of the structure tied into the base of the dam and, second, to make a heavy rock-fill against the downstream face of sufficient section to hold the broken dam against further sliding. If the company owning the dam never informed Mr. Hatton whether it adopted these plans, it certainly indicates that it did not then regard him as at that time responsible for the work. As to what engineer, if any, was charged with the responsibility or whether the whole matter was treated as one that any "practical" man could safely handle, we are not now informed. Certainly the record is an eloquent illustration of how *not* to do engineering work.

Inspection of the dam by a member of the editorial staff of Engineering News who went to the site as soon as news of the disaster was received, shows that nothing whatever was done to strengthen the dam and it was in use up to the time of its disaster with a head of water against it usually amounting to about 40 ft., according to the statement made by the President of the company owning the dam.

The question of most importance to the public is not the placing of blame for this particular disaster; but how similar disasters may be prevented in the future. Such disasters will occur and occur with increasing frequency we may be sure so long as a private individual or firm or company has the right to build a dam when and where and how he or it pleases, wherever land for a dam site may be purchased.

Twenty-two years ago, when the failure of an insignificant dam above Johnstown, Pa., wrought a disaster which shocked the civilized world, we declared in these columns that the time had come for establishing state supervision of dams and dam construction in the interest of public safety.

But the lesson of Johnstown fell on deaf ears. In the state of Pennsylvania to-day it is nobody's business to prevent such dams as that at Johnstown or that at Austin from being constructed, or to see that dams once built are so cared for as to prevent their being a menace to the valleys below them.

The State Water Supply Commission of Pennsylvania, created in 1905, has a limited authority with respect to obstructions placed in "public or navigable streams," the laws under which it is organized having in view chiefly the protection of navigation; but the Commission has no control whatever over dams on the non-navigable rivers of the state, as we are informed.

And Pennsylvania is no worse than the great majority of its sister states. Rhode Island, Connecticut and Massachusetts in the East and Colorado and possibly some other of the far Western states where irrigation is important are the only states we know of which exercise even the mildest sort of supervision over the building of dams.

Even in these states, however, the state supervision is generally exercised in an inadequate manner. We are well within bounds in saying that there is not a single state of the Union where the design and construction of dams is under sufficient control of a high class engineer acting under state authority and responsible for the public safety.

Strong arguments have been brought forward during the past few years in favor of State and Federal control of water-power development to the end that public rights in the value of such undeveloped water powers may be preserved. But considerations of public safety furnish even a stronger argument for the establishment of some sort of state control over structures which may constitute a menace to hundreds and thousands of people, and to millions of dollars worth of property. Austin and Johnstown and Mill River are far from being the only cases where terrible destruction has been wrought by a dam failure. Smaller structures fail every little while and the general public hardly notices the item of a few lines in the newspapers that tells of lives lost and homes destroyed.

Nor is it failures of dams alone which show the crying need of Government supervision of some sort to protect the public from the results of tampering with the rivers of a country by irresponsible and incompetent concerns. A glaring example is the situation of the Imperial Valley in the Colorado Delta. The story has been told in these columns how the hazardous experiment of drawing irrigation water from the Colorado without a headgate, set in motion forces which have already made necessary the spending of several millions of dollars for their control, and which have still potential possibilities of calamity.

But it may be said that state supervision of dam construction would be a farce; that the office would be a political sinecure, and that no real public benefit would result. But if this argument be sound, it applies to each and every one of our governmental activities.

Carried to its logical conclusion it would abolish city inspection of buildings and elevators, state supervision over water supplies and sewerage systems, Federal inspection of steam vessels and their machinery, of locomotive boilers, of explosives, and so we might continue through a very long list. The simple fact is that under our modern complex civilization dangers innumerable threaten on every hand if the technical work on which modern industry depends is not done with skill and fidelity.

Engineers will heartily welcome the establish-

ment of state bureaus to supervise engineering construction, provided such bureaus are officered by men of high standing in the profession and are maintained free from graft and political influence. Such bureaus in fact would be a great aid to the engineer who is desirous of doing only honest and safe work and finds himself hampered by the ignorance or parsimony of his employer.

Concerning Inadequate Salaries to Instrument Men on Surveys.

We print in our correspondence column this week two letters complaining of the inadequate salaries paid to the engineers engaged in field work on railway and other surveys. One correspondent thinks the "employing engineers," by which we presume he means the Chief Engineer or his representatives, ought to pay higher salaries to the instrument men and select only college graduates for all jobs above the grade of rodman,—at least we so interpret his letter.

Now, what does the Chief Engineer say to that? He says, we are very sure, that he is himself only an employee, like the men in the field. That he has to look out for his employer's interests and hire men to go out in the field at prevailing prices. If he pays higher salaries to his men, than equally competent men can be secured for elsewhere, he will not hold down his job very long. Admittedly he ought to try to secure competent and trustworthy men, since incompetent men who cannot be trusted are dear at any price, but there appears to be an abundance of competent young men willing to get out in survey parties for very small salaries, and a comparative scarcity of jobs in that class of engineering work.

It is not only that the engineering schools have so trained a lot of young men that they are competent to perform work of this sort. There are a very large number of men with a good common-school or high school education who have gained experience enough with the compass and the level and the transit to do good work.

There is no disguising the fact that in order to command a high price in the business or professional world to-day, a man must possess a much higher degree of skill and of technical knowledge than sufficed to give a man very creditable standing thirty or forty years ago. This applies to other occupations as well as engineering. In the machine shop of forty years ago the workman who could deal with thousandths of an inch and use surface plates and gages and high-class machine tools was an important man who could earn high wages. To-day green hands off the immigrant ships are taken in and taught how to tend an automatic machine; and fine measurements to gage are a matter of course in the most ordinary shop.

A physician of the old-time had the simple task of treating disease with empirical old remedies, and looking resigned to the inevitable if the patient died. The modern physician has seen all the comfortable empiricism of a quarter century ago swept out of doors. It is up to him to diagnose dozens of diseases which his predecessor never heard of and to undertake their cure after modern science has assured him that prevention and a certain amount of "assisting nature" are about the only really effective things which his profession can accomplish.

So we might continue to show that in many occupations besides engineering, work which ranked as difficult and highly paid a quarter-century ago is now taken as a matter of course, and plenty of people are eager to undertake it at a low salary.

To put the situation even more plainly: So far as the use of engineering instruments in accurate measurement is concerned, it is pretty clear that that alone does not constitute a valid claim for high salaries. It is what goes with it,—and more than anything else the good judgment and the fidelity to trust under high professional standards—that justifies high salaries to the engineer or the surveyor.

To illustrate: A man wants a city lot surveyed. He is willing to pay a good fee to the surveyor, not because there is anything especially difficult in measuring a city lot to a fraction of an inch; but because he must trust to that surveyor's honor that the work is done with accuracy and fidelity. Or perhaps it is a tract of wild mountain land where the large experience of the surveyor in retracing old lines and long lost corners is worth a good price.

But where a man is an employee of a large organization and his work is inspected and checked, he is expected to do reliable work anyway, and is not paid extra on that account, whether he is making accurate measurements in the machine shop or in the field.

Under these conditions "employing engineers" are, it seems to us, practically powerless to raise the prevailing rate of wages. Our correspondent charges that a penny-wise policy is often followed by hiring cheap men who have not a college education. But it will hardly be claimed that only engineering school graduates can do accurate and reliable work with transit and level. Of the men who surveyed the railway lines built prior to 1880, very few had ever seen the inside of an engineering school. The engineering-school graduate to-day must meet the competition of men who have been working while he was studying; some of these men are high-school graduates; many have taken courses in the correspondence schools. The graduate of to-day knows pretty well that his degree counts for nothing in the competitive struggle. If he wins it will be because he has gained in winning his degree, something his competitor lacks.

Another correspondent, who has shaken the dust of surveying parties off his feet for good advises that engineers form a union and demand union wages, and we presume union hours.

This is not a new suggestion. Many a hard-working engineer has contrasted his own twelve-hour a day job plus lying awake nights planning how to solve tough problems with the job of the eight-hour trade-union workman who came over in the steerage less than a dozen years ago, and yet earns nearly as much money, year in and year out, as the engineer.

It is not strange at all that engineers who find themselves being ground in such a mill should come to think that labor-union methods are the only means by which they can get fair compensation for the work and the responsibilities laid upon them.

But the day the engineering profession becomes a labor-union, it will forfeit its standing in the public estimation as a profession. And while it is common to hear engineers declare that engineering is only a business anyway, the great majority of engineers, we believe, still take pride in their occupation as a profession, and a profession deserving of more honors and higher emoluments than it has yet received.

It needs but the slightest study of the subject to see that the organization of engineers in a trade-union would be impossible even if it were desirable.

A trade-union can only control conditions in an industry when it can bring into its membership a substantial proportion of all the workers in the industry. Trade-unions can be effectively organized in industries which are concentrated in cities and towns. They fail in industries which are scattered all over the country. The trade-unions have found it necessary in order to maintain their position to limit the number of apprentices. Could engineers limit the number of men who shall be taught engineering?

There is this to be said, however, for the trade-union idea: That in so far as it signifies co-operation for mutual benefit, it is emphatically the right and the duty of engineers to effect this. There should be more cooperation among engineers with respect to the formulation of standard scales of fees, for example. Work in this direction has recently been recorded in these columns. In our issue of Aug. 17 we discussed at length the question how the engineering profession through its existing organizations could cooperate to better existing conditions in the marketing of engineering services.

So far as "union" signifies cooperation for mutual benefit in a manner calculated to merit public approbation and increase the respect in which the engineering profession is held, it has our most hearty approval and support.

More than that; we would urge that every engineer, from the lowest rank of the profession to the highest should feel that he owes a duty to his profession as well as to himself. One reason why engineering compensation is low is because there are so many who are willing to accept inadequate compensation for their work. There are engineers, on the other hand, who have reached a degree of eminence and independence that enables them to charge high rates for their professional services and to refuse work unless they can be paid such rates.

Other men in a more limited field have been known to declare that rather than work at engineering on a less salary than the labor and responsibility justified, they would go back to the farm or the shop. If there were a large enough proportion of the profession able and willing to take such a stand, the question of engineers' compensation would settle itself.

LETTERS TO THE EDITOR.

Concerning the Compensation of Men on Railway Survey Parties. One Reason for Low Salaries.

Sir: I have read with considerable interest the letters in your columns at various times pertaining to the compensation of engineers and graduate engineering students. The frequent appearance of such letters makes it evident that the profession is conscious of the inadequate compensation received for engineering services. As my observations have been very discouraging in this matter I have tried to solve to my own satisfaction the reason for present conditions and the possible path to betterment. My conclusions are that the employing engineer is largely, if not wholly responsible for the present existing low standing of the profession, and its inadequate low compensation for services rendered.

I wish to present some of my observations and conclusions. In doing so, I shall deal with the more subordinate positions, for it is here that I have been able to see the most flagrant abuse, and too, each engineer of the future must pass this point in his career.

My experience covers railroad work, power development and water supply and is a well balanced mixture of office and field work. In each line I have found conditions practically the same.

On a railroad surveying party of fifteen men I have often found myself the only college man. The balance of the party, would be pick-ups, educated to their present state of usefulness at the enormous expense of the companies for which they had worked, often gaining their positions by deceit in their representations to the man hiring them, and often by the false idea of economy on the part of the employer.

We often hear the remark made that there is only one man in many who fits the position for which he was hired. And why is it? For an answer let us analyze the situation cited above where most of the party men were not trained for their work, but are merely rustics who have gained their positions by misrepresentation and ways best known to themselves. The possible path opened to them is as follows:

A Chief of Party back in the mountains and remote from any large city finds himself minus a leveler. He sizes up the situation something like this:

1.—This work is about completed and is only a question of a few weeks.

2.—It is impossible to procure some one to take his place in less than three days and possibly more.

3.—Perhaps Mr. Rodman, who seems to be an intelligent chap can fill the position, with much assistance, temporarily at least, or until the work is completed. So Mr. Rodman is made leveler, Mr. Chainman is given the rod and there is a movement along the line, and the vacancy in the ranks of axemen is easily filled by a native of the vicinity.

A proper and just advancement many will say. But is it? To begin with, the new leveler was not fitted for any such advancement in responsibility and he has to be trained for the new position, as well as many others who were advanced before their time. Perhaps in so far as this particular party is concerned, considering that its work was about completed, it is as well off by its

present crippling as it would have been had it been held up waiting for a capable man from the outside.

But the damage to this particular party is not the whole damage that is done. The real damage is to the profession and comes later in this way. Each of these men advanced before he was ready for the advancement is now open for employment, the work of this party being completed. In seeking employment each aspires to secure a position as good or better than the one he has just left. As an example let us follow our newly made leveler. In his search for a new position Mr. Leveler enters an office where a party is being made up to send into the mountains, perhaps to some very remote section where it is difficult to get word in and out. A leveler is needed and a hearing is given. Mr. Employer learns that Mr. Leveler has just left the employ of Mr. Chief, the work being completed. Knowing Mr. Chief to be a good engineer and conservative in his choice of help, Mr. Employer considers it a good recommendation to have held such a position with him. Mr. Leveler of course does not mention the short period that he held the position, nor the rechecking and assistance that he got continually from the Chief, nor the circumstances which forced him into the position.

Mr. Leveler is hired and the party goes to the mountains. After a week has passed the poor quality of the new leveler's work is brought to the Chiefs notice, and the work of the whole party for that time is practically useless. A rather expensive leveler isn't he?

This case is not at all exaggerated. Suppose the party were running to maximum grade, an error or errors could make the line absolutely valueless and the entire line would have to be re-run, and the Chief might consider himself especially fortunate to have caught the error so soon. This expense is borne by the employing company.

Again the Chief has a problem to solve. To send for a new leveler and run the risk of getting one who is no better and be crippled while waiting for him or to endeavor by extraordinary diligence to break in the one at hand. Again time and distance work wonders for the new leveler and he is allowed to remain, to the exclusion of a deserving and capable man, a lowering of the standard of the profession and a reduction in its compensation. This leveler's education is an expensive one to which we must all contribute in the end, and for which the employing company is heavily taxed at the present time.

The writer does not mean to imply that this is always the case for there are many exceptions, yet it is generally true.

This of course is only a single illustration, but the writer can enumerate many where men have been trained from the ranks of the ineligible to fill the position of recorder, assist in the drafting room (temporarily as is usually supposed) and numerous other positions that should be filled from the ranks of those who have prepared themselves for such work. These positions may be only temporary but the damage is done. The germ of discontent is rooted, a false idea of their ability is gotten that is probably never to be eradicated and thus we have a floating multitude of "would-be's", misrepresenting themselves to those to whom they apply for employment, quite generally deceiving the public at large, and by the quality of their work dragging engineering as a profession through the mire of public contempt.

I might go on indefinitely but the above I think is sufficient to illustrate my point. The question is, what is the remedy?

To the writer's mind, the remedy lies in the hands of the employing engineer. He is today largely responsible for the present state of affairs. If he would be more conservative in his selection of men, hire only those who are competent and pay them what they are worth, then we would soon see a decided change in the standing of engineers. I wish that "Give the engineers a chance," could be made the slogan of all engineers. If engineers could only be made to see a few months into the future instead of only the crying needs of the immediate present then much good to the profession could be done.

In the trades, the unions limit the number of apprentices. In the field of engineering a different view is taken. Every engineer is glad to see an ambitious young man endeavoring to better his condition, and is usually aiding him in every way possible. This generosity is to be commended, but we should look at it and its results fairly.

When a student at college, I endeavored to find employment on survey parties for the summer vacations, but I could not find such a position. Since I have been in the work I have floundered around all summer with farmer boys and men who were disinterested, endeavoring to accomplish something, but practically to no avail. I have had parties totaling twenty or more men where not a single man was a college graduate. The result was disastrous. Many a month has passed with 35% of the working time put in re-running our work. Why? Because the office insisted upon supplying the men and prompted by false ideas of economy hired the cheaper men, being able to get them from the ranks of the "would-be's" for less than must be paid for an efficient man.

A man who has put four or more years of the best part of his life in preparation at an expense of several thousand dollars can not compete in price against one who is being educated at the expense of his employer. The result is disastrous from every point of view. The employing firms are paying far too much for the results, the price on engineering talent is lowered and the "would-be's" are starting on a tangent which often leads them into a blind alley.

If the employing engineers of today could be made to realize that those who have prepared themselves should be given the preference, then the standard of the work would be higher, the quantity of work accomplished be greater; then the public would be made to realize the real value of engineering and the engineer of tomorrow would get his just compensation, and the engineering profession would naturally take its rank among the other professions.

To be sure we will always have with us the engineer who seeks to make his position the stepping stone to his own independence through channels that can not be rightly called engineering; who uses his authority as a lever to boost some deserving man from a position that he efficiently fills to make room for some relative, near relative, or friend.

This is to be deplored, yet I can not but think that there are plenty of honest engineers in the field, so that by cooperation in action along conservative lines in the employing of the assistants the desired results could be obtained in another generation.

Yours truly,

A Subscriber.

San Francisco, Cal., Sept. 6, 1911.

Sir: Before I begin to write on the subject I have at heart, I will tell you something about myself: I obtained my engineering education at the Virginia Military Institute. My first engineering work was for the U. S. Coast Survey as a rodman, 1889-90, at a salary of $40 per month, promoted to levelman at $60 per month. From 1890 to 1900 it was my lot to receive for my work about such compensation as I have outlined above.

I venture to say that 90% of the active working engineers and surveyors in this country today do not earn $1,000 a year. The mechanics working under the direction of the engineers in many cases receive twice the pay of the engineers over them.

My object in writing this letter is to start a movement that will change these conditions. Not that I myself hope to reap any benefit from the change should it be brought about, for I now have the means to provide my own employment and thank God am no longer looking for any old job that will provide board and clothes.

I know to my certain knowledge that there are thousands of surveyors and engineers who are earning just a bare existence. Is it not possible for Engineering News to inaugurate a movement that will mean the Unionizing of all the surveyors and engineers in the United States with an established schedule of compensation for every position? We must either do this or be forced to the wall.

When transcontinental railroads are able to hire competent men for $50 to $75 per month, force them to furnish surveying instruments and work them 14 hours per day, it is certainly time to call a halt. Do take this matter up for sooner or later we must show the world that we are as brainy as Hod Carriers, Bricklayers and Stone Masons.

B. N. T.

Los Angeles, Cal., Aug. 10, 1911.

[We have commented on these letters in the editorial columns of this issue.—Ed.]

The Cost of Preliminary Railroad Surveys in Mexico.

Sir: In the article in your issue of Sept. 7, 1911, describing the country through which the proposed Puebla & Pacific R. R. is to run, it is stated that the **preliminary surveys** of the line of this railroad were made at a cost of about $32 gold per mile. This is obviously a mistake.

As the term is generally understood among American engineers, the preliminary survey involves not only the running of a fairly accurate transit line and levels, but also the preparation of a topographical map of a strip of country through which the final location will pass, on which the best line the country affords under the governing conditions, can be projected or laid out in such a manner that it can afterwards be reproduced on the ground.

Such a survey and map, if properly made in the kind of country described in your article, will cost in Mexico at least as much as it will in the United States, and considering the difficulty of the topography, the fact that there are no maps, and that the country is to a certain extent unexplored, it is safe to place this cost at not less than $150 gold per mile and it might easily be $250. This is assuming competent engineers at the ordinary salaries such men command in Mexico.

I think it would be of interest to your readers, as I know it would to me, to know just what was accomplished for $32 gold per mile or whether this is an error.

F. Lavis.

50 Church St., New York City, Sept. 9, 1911.

[Proof of the above letter was submitted to Mr. Roberts and we append his reply.—Ed.]

Sir: In reply to Mr. Lavis's inquiry, the party consisted of a chief of party and four men, whose salaries were as given in the accompanying tabulation and distribution of field expenditures for the month of January, 1911.

As will be seen, these expenditures do not include the salary of the chief engineer, nor his office staff; only those for the field force.

Where the topography was very steep, requiring time and ropes to take it, the contours were only sketched in by eye, measurements were taken with a tape line only to the foot of the canon walls, it being the intention to run a more accurate preliminary line through this canon before location was started.

In all other places, however, the topography was taken by the usual tape-and-hand-level method, giving a belt of topography from 100 to 300 ft. wide on each side of the center line of the survey on which the best line can be projected and afterwards reproduced on the ground.

The party was generally on the ground at 6.30 a. m., the chainmen being on the run all day. When possible the work was continued till 6 p. m., with one hour out for lunch at noon.

If the topography was more than one day's work behind the transit line, the whole party would turn in and lend a hand at taking topography, every one plotting it up at night. When the topography was within one day of the transit line, camp would be moved ahead.

While the work cost $30 gold per mile for the month of January, 1911, the average cost was about $32 gold per mile for some 150 kilometers of line. Yours truly,

H. M. Roberts.

Oaxaca, Mexico, Sept. 18, 1911.

P. & P. R. R. Co. Field Expenses January, 1911.

Position.	Salaries. (Mexican currency).
Chief of party	$325.00
Transitman	250.00
Levelman	200.00
Topographer	225.00
Draftsman	200.00
Cook	60.00
Total	$1,260.00

Distribution.

	Kitchen.	Stable.	Surveys.	Traveling expenses.	General expenses.
By purchase.	$152.77	$93.43	$34.88	$86.85	$33.09
Pay roll	15.50		296.32		15.50
Salaries	60.00	$1.00	1,200.00		
No. of persons or animals maintained.	6	23			
Cost per mo. per person or animal.	$33.05	$5.41			

Total expenditures of field force for month $2,025.34

Number of kilometers of line run during month 54

Cost per kilometer (Mex. Cy.) = $\frac{\$2,025.34}{54}$ = $37.50

Cost per mile U. S. Cy. = $30.

Concrete Casings Filled With Sand as Wooden Pile Protection.

Sir: As there lately has been a great deal of discussion up and down the Pacific Coast whether concrete will be affected by the salts in the sea water, and a good many examples have been quoted where such seems to have been the case, it may be of interest, and may lead to a little more closer study of the reasons for such happenings and a general discussion on the sub-

FIG. 1. WIRING REINFORCEMENT FOR PILE CASINGS.

FIG. 2. CONCRETE PILE CASINGS IN YARD.

ject, to put on record our experience with concrete piling here at Humboldt Bay.

We have here about 15 miles of water front, where the depth is such as to require piling from 35 to 40 ft. long. Of course, as this is the center of the lumber industry on the Pacific Coast, and the biggest part of the water front property is owned by the mills, the wooden piles can be furnished and placed at a minimum cost. The teredo, however, raises general mischief with this class of construction. The writer has seen 15-in. pine piles eaten entirely off in eight months' time. This is exceptional, the average life of a pile being estimated at about two years. For outside dock work this is not so bad, as the cost of the piles are so low, but where there are warehouses on the docks, it alters the conditions considerably, as the redriving interferes with the traffic,, to say nothing of the tearing up and rebuilding of warehouses.

For this reason, a permanent construction would be a great relief, and about four years ago the writer started an agitation for concrete piling. Records were looked up and investigations made where similar construction had been used, but, as stated earlier, it seemed as if concrete had not been very successful for this class of work. On the other hand, the writer knew that right here in the bay, concrete pile protection had been placed around wooden piles over 15 years ago, and that, even at this date, it shows no deterioration. It could be seen here, and of course local demonstration is better than foreign news.

The concrete referred to was placed very crudely, by the Northwestern Pacific R. R. Co., and consisted of driving a 1-in. sheet piling around the wooden pile, pumping the dirt out about one foot below original grade, and filling the whole thing in with 1:2:3 concrete, at low tide. The forms were left on until after a couple of months, when the teredo had eaten the wood away, leaving the concrete exposed. Up to the present time, the concrete stands without any visible signs of wear. On the strength of this demonstration, the writer closed up a contract for a number of concrete pile protections, and the following design was devised:

A ball and socket joint cylinder, 22 ins. inside diameter, was manufactured in 10-ft. lengths, with 3-in. walls. From Fig. 1 the reinforcing method can be seen; ¼-in. round rods were used for verticals, 5 ins. c. to c., and No. 9 black wire, 5 ins. c. to c., for horizontal hoops. The cylinders were not figured to carry any weight, or to be exposed on the outside of the dock, as wooden fender piles were driven here; the only purpose for the reinforcement was to prevent cracking, and enable the castings to be moved from the yard to their respective places around the wooden piles.

Manufactured wire netting was first used, but it was found in a very short time, that we could shape and tie our own reinforcement a great deal more satisfactorily and with less expense than the wire netting could even be bought for.

The wire hoops were made to exactly the right diameter, slipped over a drum with removable end, and wired at all intersections to the ¼-in. vertical bars. The reinforcement was placed in the center of the 3-in. wall. Wooden forms were used and found to be very satisfactory. The inside core was made in four parts, with a key that could be released by removing a couple of pins.

The outside forms were made in three sections, bolted together as chimney forms. The piling were made standing on end, and a very wet mixture used, proportioned 1:2:3; coarse river sand and screened gravel, ranging in size from ¾-in. to pea size. Waterproofing in the shape of soft soap was used in the mixing water, and about 2% of lime was used with the cement. The waterproofing was added merely as a precaution, and to make the concrete as dense as possible, so as to prevent the water from entering into the concrete.

The forms generally were removed the day after being poured, and the piling cured and kept wet for three weeks. No piles were moved and placed before being six weeks old, at least. A home-made continuous mixer was used, and between 25 and 30 piles made each day.

It was found, in moving the piles, that they would stand extremely rough handling. The pile-driver would throw the snub line over them and drag them over the ground for considerable distances, without any sign of cracking, and after being placed around the wooden pile, they were driven down in the ground about 2 ft. 6 ins., without any trouble. Two casings were used in most instances, cemented together at low tide. After the casings were placed, there was left about a 3-in. space between the wooden pile and the concrete. This space was filled with wet sand, placed by a sand pump, and it was found that this would pack as solid as rock.

The biggest part of these pilings are now in place, and everybody is very much pleased with them.

Thos. Engelhart.

Eureka, Cal., Sept. 5, 1911.

Sluice-Gates in the Rock Tunnel of the San Luis Valley Irrigation District, Colorado.

Sir: I note in your issue of Aug. 17, a description of a large sluice-gate for controlling the flow of water through a tunnel; it occurs to me that some work of this kind recently executed in Colorado may be of interest to your readers.

The San Luis Valley Irrigation District is constructing a dam across the Rio Grande at what is known as Thirty-mile Bridge, in Hinsdale County, Colorado. This dam is a combination rock and earth fill with a maximum height of 100 ft., and will form a reservoir impounding about 43,000 acre-ft. of water.

The outlet from this reservoir is through a tunnel in the rock having a length of about 720 ft. and a cross-section 11 × 15 ft. Near the middle of this tunnel there is a shaft connecting it with the surface, and through this the sluice-gates controlling the discharge are operated. These gates, which are shown in the accompanying illustrations, and the structure which supports them, are the most interesting features of the work.

There are two complete sets of gates, 4 ft. 3 ins. apart, operated independently, either of which may be used for controlling the discharge. Each set consists of five gates, each gate presenting a clear opening of 3 × 8 ft. The gates are of cast iron running on cast-iron guides provided with bronze liners. The supporting framework is of structural steel. All open spaces above, below, and around the steel work are filled with concrete carefully tamped into place. The structure contains about 125,000 lbs. of structural steel and an equal weight of cast iron.

While it is not contemplated that water will ordinarily be discharged from the reservoir at a rate corresponding to the full capacity of the tunnel, it was necessary to provide a very large tunnel in order to pass the entire flow of the river, which at times exceeds 2,000 cu. ft. per sec., during the construction of the dam. The greater portion of the work will, of course, be done while the river is at a low stage.

The reservoir is constructed to store water for irrigation purposes only, and the work seems to be worthy of notice because of its substantial and permanent character, and the foresight that has been displayed in providing for all possible emergencies. This is in strong contrast with the ordinary practice in irrigation works, where temporary make-shifts are installed instead of permanent works, and everything is trusted to luck. When accidents happen to such works, as they do three years out of five, the troubles are blamed on the "Yellow Peril," the Mexican insurrection, or reciprocity with Canada!

The Rio Grande reservoir was designed by and is being constructed under the supervision of Mr. J. C. Ulrich, M. Am. Soc. C. E., of Denver, Colo.

Charles W. Comstock,
State Engineer.

State of Colorado, Engineering Department, Denver, Colo., Sept. 2, 1911.

Fig. 1. Inlet Side of Sluice-Gates.

Fig. 2. Outlet Side of Sluice-Gates.

FIGS. 1 AND 2. SLUICE-GATES OF THE RIO GRANDE RESERVOIR, COLORADO.

Notes and Queries.

In the description of the Southern Pacific **"roller-case" pile-driver**, p. 338, of our issue of Sept. 21, 1911, several errors crept in which should be corrected, as follows:

The lengths of the embankment approaches (4th to 6th lines) should read, "5.9 and 2.5-mile solid embankment approaches from the west and east shores respectively." The text as printed reversed the lengths and also credited each with 0.1 mile less.

The grade of approach is 0.2%, instead of 0.1% (4th paragraph).

The trestle extension, which was stated to be 3,000 ft. westward, should be 4,500 ft. westward; this correction is to be made at two places in the text (2nd col., 2nd paragraph; 4th col., 3rd paragraph).

The time of completion of the extension work should read 2½ months' time, instead of 2 months' time (4th col., 3rd paragraph).

A System of Standard Benchmarks in New York City.

In the past there has been no unified system of benchmarks or reference points for levels in New York City. Worse still, the various city bureaus having surveying to do employed each its own individual datum-plane or zero elevation. About half a dozen years ago the Society of Municipal Engineers of New York City undertook an accurate comparison of the several datum planes largely used in the city.* Later, the society recommended that all city bureaus and departments refer their elevations to Geodetic Survey datum, namely mean sea level at Sandy Hook.

More recently the society induced the city authorities to undertake the work of establishing a system of benchmarks throughout the city, and run precise-level lines to fix their elevations accurately. An appropriation of $10,000 was made, subsequently increased by $5,000. The work was placed in the hands of the Engineering bureau of the Board of Estimate and Apportionment, Mr. Nelson P. Lewis, M. Am. Soc. C. E., Chief Engineer. Mr. Frederick W. Koop was placed in charge. Work was begun late in 1909, and is still going on; it will be completed presumably early in 1912.

A good concise description of the work is contained in Mr. Arthur S. Tuttle's presidential address, "Proceedings of the Municipal Engineers of New York City" for 1910, just issued. We quote farther below from Mr. Tuttle's description.

In its administration, the work was to some degree continuous with the precise triangulation of New York City, begun seven or eight years ago in charge of Mr. A. T. Mosman (U. S. Coast & Geodetic Survey), under a cooperative arrangement between the city and the Geodetic Survey. Mr. Koop was engaged on this triangulation, and later applied Geodetic Survey methods to the precise-level work for the benchmark system.†

Quoting from Mr. Tuttle's address:

As it was desirable to have the work conform with the standards used by the U. S. Coast and Geodetic Survey, orders were placed for the necessary precise-levels and other instruments, all of which were of the latest U. S. Coast Survey type and made from specifications furnished by the instrument division of the Survey.

A reconnaissance was then made over Brooklyn, Queens and Richmond, level lines selected and a level-net planned for these boroughs. These level lines include all the old U. S. Coast Survey benchmarks recovered, and the benchmarks of the various city departments, about one to each mile.

In selecting and preparing the benchmarks to be established along these lines, the type of benchmark found to be most permanent and in use by the various governments and surveys in large cities was adopted. For primary benchmarks this type consists mainly of brass tablets suitably inscribed and set in the solid masory of substantial public buildings; a secondary point, just as accurately determined but somewhat more convenient to hold upon, accompanies the primary point. For suburban districts a monument set in a substantial mass of concrete, the bottom of which is well below the frost line, is used for primary points.

In the outlying and unsettled parts of Queens Borough twelve granite monuments, to be used both as traverse points and as primary benchmarks, were set in concrete, and it is proposed to set such monuments in each of the remaining boroughs. The large monuments marking the U. S. triangulation stations and the section ends of the base-line are also used as primary points. Brass, bronze, copper or other metallic projecting benchmarks likely to be disturbed or stolen have not been set in connection with this work.

Up to date [Jan., 1911] over 300 benchmarks of various kinds have been established in the three boroughs, and a total of 215 miles of completed level lines (or over 430 miles of single line) actually covered in these three boroughs.

The mean rate of divergence between forward and backward lines on the entire work has been, up to date, a very small fraction of the divergence allowed on the national surveys. The metric system is used on this work, and the final results, when published, will be shown in meters and in feet. This work is comparable with the best precise-leveling done in this or in any other country, and when the work is consummated, the City of New York will have a comprehensive system of primary benchmarks covering each borough, all referred to the one universal datum plane adopted by the engineering departments of the national government, mean sea-level at Sandy Hook.

The figures given above have by September, 1911, grown to the following, as stated approximately by Mr. Koop: Number of benchmarks established, 380. Length of precise-level line run, 255 miles of double line or 510 miles of single line. About two-thirds of the benchmarks are new ones set specially for this work, while the others are previously existing benchmarks of bureaus, etc.

The entire work has been carried on by a single field-party. The progress made is described in the last annual report of the Chief Engineer of the Board of Estimate thus:

The system of benchmarks in the boroughs of Brooklyn and Queens has been completed, as has also been a large portion of the work in the Borough of Richmond. A reconnaissance has been made covering the boroughs of Manhattan and Bronx, locations for a number of the benchmarks have been selected, and the detail work will be done during the coming year. The benchmarks established are permanent in character and are placed either upon substantial buildings or special monuments set in concrete have been located wherever possible upon land owned by the city. Wherever it was possible to do so, benchmarks have been established in pairs, so as to insure the permanency of at least one of them.

Further notes on methods and results are contained in the report of the engineer-in-charge (Mo. Koop), which is appended to Mr. Lewis' annual report.

The primary level lines almost completely encircle the two boroughs of Brooklyn and Queens, connect one with the other, and traverse them in every direction. One of the primary circuits in Queens has a perimeter of 52 miles, and the closing error is 1 cm. The primary lines extend from Fort Hamilton to Willets Point, a distance along the level-line of 28 miles, and from the East River at Long Island City to Belle Harbor, Rockaway Beach, a distance of 27 miles. A continuous line from the U. S. Coast Survey tide staff at Fort Hamilton connects Brooklyn, Long Island City, Flushing, Little Neck, Floral Park, Queens and Springfield, giving a continuous line over 40 miles in length, connecting at Springfield with the Long Island City and Rockaway Beach line.

The work in Brooklyn and Queens has been remarkable for the high degree of precision attained without any additional expenditure of time, and the results attained are most gratifying. It was also very interesting to discover by direct connections in the field that the result of 16 years' continuous tide gagings by the U. S. Coast Survey at Fort Hamilton shows that mean sea level at Fort Hamilton is the same as at Sandy Hook.

The Borough of Queens gives the largest level circuits, and the North Shore circuit of 40 miles shows a closing error of 6 mm., or 0.0193 ft., or 0.000495 ft. per mile. The Flushing, College Point, Whitestone, Willets Point and Bayside circuit, with a perimeter of 11 miles, shows a closing error of 0.10 mm., or 0.00033 ft.

In the Borough of Richmond the system of levels consists of a primary circuit running almost entirely around the outskirts of the borough, having an approximate perimeter of 45 miles. This circuit has been connected on the northwest with the U. S. Coast Survey bench at Elizabethport, N. J., on the east with the U. S. benches at Great Kills, Fort Wadsworth and Fort Hamilton, and will shortly be connected at the Raritan River on the south with the U. S. Coast Survey transcontinental level line from Sandy Hook, N. J. This largest Richmond circuit is divided into five smaller circuits, one of the dividing lines passing through the highest land in the borough.

In the Borough of Manhattan the level plan consists of a primary line running the entire length of the borough, with primary lines covering the territory on all sides of Central Park. The main line connects with U. S. benchmarks at Governors Island on the south and with the Fort Hamilton to Willets Point line on the southeast. It continues north through the Bronx, Yonkers, etc., and connects with the U. S. primary benchmarks at Dobbs Ferry, N. Y.

In the Borough of Bronx the level-lines encircle the entire borough and much of the territory north of the city line, and join the Manhattan primary line with the Fort Hamilton and Willets Point line across Long Island Sound by way of Throggs Neck and Fort Schuyler. Connections are also made with the Manhattan line at several places between the Third Ave. bridge over the Harlem River and Yonkers. The larger traverses have been divided into smaller ones by lines running through country in which the most permanent benchmarks could be placed.

In selecting and preparing the benchmarks to be established along the primary level-lines, the type of benchmark found to be most permanent and in use by the various governments and surveys in large cities were adopted. Primary benchmarks consist mainly of brass tablets suitably inscribed and set in the masonry of substantial public buildings; this type of benchmark is very permanent, not difficult of access, and easily recognized. A secondary point just as accurately determined, but somewhat more convenient to hold upon, is placed near the primary point.

Large copper bolts were also set in solid masonry, principally in the more massive buildings; they are very permanent and are readily identified. These marks have been supplemented by cuts and marks on bridge abutments, foundations, etc. Monuments have also been set in concrete, well below the frost line.

Wherever possible, benchmarks were established in pairs, particularly on public buildings, so as to insure the permanency of at least one of them.

Some time was also given to consultations with engineers from several of the boroughs with a view to the establishing of a uniform and permanent linear standard. This standard of linear measure should obviously be the U. S. standard. While the engineers of all the boroughs are emphatic in stating that this is their standard, several of the boroughs have serious difficulty in making their traverses check between the triangulation stations. A measure of the Ocean Parkway Base-line by some of the engineers shows discrepancies between their measures and the U. S. Coast Survey base-line measurement amounting to more than ½-ft. per mile.

It is expected that the final cost of the work will remain within the appropriation of $15,000. All charges of the work except the salary of the engineer-in-charge, who is merely delegated to the work from the regular staff of the Board of Estimate, are paid from the appropriation.

*Their results were quoted in an editorial article, "Records of Datum Planes," in Eng. News, May 17, 1906, p. 543.

†Some of the final triangulation work is also being carried out by his present office, which bears the title of Division of Triangulation and Precise-Levels, Board of Estimate and Apportionment, with quarters at 26 Court St., Brooklyn.

A Pole-Preserving Machine.

It has been recognized for a long time that the destruction of wood telephone, telegraph and transmission-line poles was largely near the ground line where existed a comparatively narrow zone favorable for the production of the rot bacteria. It is often considered sufficient to apply preservative to this portion of pole only, and this has been done by brushing and by impregnating the butts of the poles by the open-tank process. The brushing generally does not give sufficient protection, and the treatment of the whole butt often would be an unnecessary expense if the attacked zone alone could be saturated. Messrs. G. P. Benton and I. B. Eberhardt, after several years of experimental work, have

devised a method and a machine for injecting the preservative into the wood in a narrow belt but to a sufficient depth to insure protection.

Figs. 1 and 2 show the machine. It is about 19 ft. long and 6 ft. wide over all, mounted on wheels for drawing about. The machine comprises a steam boiler, an air compressor, air-storage tank, a closed oil tank with steam coils, and an air-tight canvas band or belt, 3 ft. wide, with which to enclose the pole where it is to be treated, together with the necessary mechanism to wrap the band about the pole and make it tight. The operation of the machine is as follows: A pole is rolled up onto the horizontal beams and the two segmental track rings are closed about the pole. Then a spindle carrying

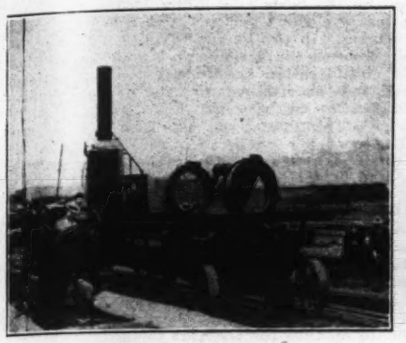

Fig. 1. As Hauled About.

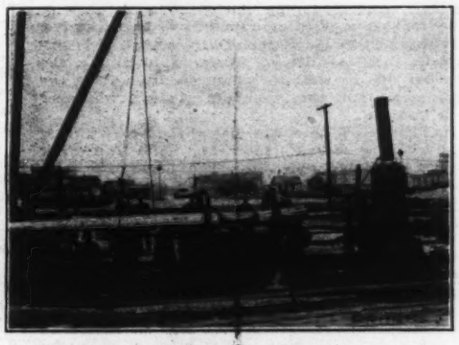

Fig. 2. With a Pole in Place.

FIGS. 1 AND 2. THE BENTON-EBERHARDT MACHINE FOR TREATING WOOD POLES.

the canvas strip, is revolved on the rings about the pole and a clamping bar converts the belt into a closed bag. Air pressure is applied to the edges of the band to close it around the pole and then the oil is forced through a pipe connection into the bag and around the pole, pressure being held on for any desired length of time. Should any oil leak by the ends of the bag, through cracks or checks in the wood, it would drip into a catch basin and might later be drained into the oil tank. By reversing the above operations, the pole may be released and taken away on the other side of the machine. Where a large number of poles have to be treated, a platform may be built on either side of the machine, or any desired construction made for ease in bringing poles to the machine. For a small number of poles in any one place, inclined skids may be placed on each side of the machine. The capacity of the machine is claimed to be about 50 poles per day. The cost of treating, including labor, oil, fuel and fixed charges, is something less than $1 for a 35-ft. pole, with a 7-in. top.

A demonstration was made on Sept. 20 in the pole yard of a large electric power company in Chicago before representatives of the U. S. Forest Service and of other electric companies. Three 35-ft. Michigan cedar poles, cut last winter, were treated 7, 10 and 15 minutes, respectively, with hot creosote oil, under an air pressure of about 5 lbs. per sq. in. With a 10-minute treatment an initial penetration of 3/16-in. was obtained, and in a few days it had apparently increased to something over ½-in. The treated belt was 3 ft. wide and the absorbed oil amounted to 1 gal.

The inventors have patented the process and the necessary equipment. The machine pictured is manufactured by the B. & E. Pole Preserving Machine Co., 2014 Fisher Bldg., Chicago, by which concern this information has been furnished.

A 2,800-Ton Ship with Diesel Engines is being built in Germany. The ship is to be 233 ft. long and 39½ ft. broad. The propelling engine will be a six-cylinder Diesel engine of about 1,500 HP. capacity. The vessel is being built for the Deutsche Dampfschifffahrt Gesellschaft Hansa by J. C. Tecklenborg Co. of Bremerhaven. This concern has for some time been manufacturing reversible Diesel engines, but none has been built as large as the engine of the new ship. [Zeitschrift des Ver. deutscher Ing., Sept. 16, 1911.]

The Development of Steam Turbine Designs.*

By EDWIN D. DREYFUS.†

The increasing volume of steam turbine construction, reaching a total upwards of 20,000,000 HP. in the short time elapsed since the advent of this prime mover, establishes its permanency as a factor in our economic power problems. Therefore, rudimentary facts of design and development deserve small space here, and the author accordingly undertakes only to outline some results of experience, upon which the choice of turbine designs should be founded.

CAPACITIES.—For electrical supply, the turbine has become commercially available in sizes ranging from a fraction of a kilowatt up to as high a power as 30,000 KW. in a single unit. The problems which enter the design vary according to the magnitude and importance of the service. Small turbines under 300 KW. necessarily assume an elementary form—usually a simple impulse wheel. Moderate sized units, from 300 to 1,500 KW., or somewhat above, are required to possess higher efficiency, and also great mechanical stability as the operation grows in importance. Still larger machines must embody the best features of the preceding class, but at the same time any involved construction due to the extent of forces and proportion of parts, must be avoided. The designer has fortunately two well known schemes of design, commonly designated the "impulse" and the "reaction," to apply, as experience may require, to the greatest mechanical and economical advantage.

A noteworthy innovation, which appeared several years ago, presents itself in the form of an effective combination of both types, termed a combined action and reaction turbine, but more familiarly known as the double-flow design.

VARIED ARRANGEMENTS.—In view of the increasing number of problems confronting the power engineer it is opportune that the steam turbine admits of wide deviation from standard working conditions.

Complete-expansion turbines essentially rank first, owing to the predominance of their use. These receive steam at the highest steam pressures and temperatures (superheat), expanding it continuously to the greatest degree of vacuum. With these high boiler pressures (ordinarily above 175 lbs. gage) but a very small part of the turbine is required to withstand the accompanying stresses, as the high pressures are confined within small nozzle blocks, unlike the case of the reciprocating engine, where the high-pressure cylinder must safely accommodate these forces. The turbine cylinders have no rubbing surfaces, so that lubrication under high temperatures is not a factor. The profitable use of a high vacuum is readily accomplished without impractical or uncommercial proportion of parts, such as would be necessary with low-pressure cylinders of reciprocating engines. Therefore, higher heat efficiency obtains with the turbine.

Low-pressure turbines up to date hold a position second to that of the complete-expansion type, since they are capable of receiving steam at approximately the pressure at which a piston engine would exhaust the steam and complete the expansion to high vacuum with the same degree of efficiency. Hence, the use of such a turbine may double the output of a non-condensing reciprocating-engine plant without increased coal consumption. Considering a reconstructed condensing-engine plant, the actual improvement will usually be from 10 to 50%.

The addition of low-pressure turbines to reciprocating-engine stations becomes important (1) in reducing unit expense of fuel, oil, repairs and labor, (2) in increasing capacity at low unit cost, (3) in improving the service by reducing the condenser equipment or providing a good supply of boiler feed water, (4) in placing the least tax on boilers at overload, and (5) in better electrical regulation.

The engineering features of a low-pressure turbine installation depend on various plant conditions and may demand (1) use without a governor, there being electrical control through the synchronizing force of the generator, (2) governor control with the admission of auxiliary high-pressure steam, (3) the interlinking of turbine and belted engine through a synchronous motor, (4) automatic by-passing of low pressure steam to condenser, (5) use of a reserve high-pressure element, (6) use of a heat-storage system.

Non-condensing turbines have a field of usefulness as main power units, but to a limited extent as compared with the other types. Should heat supply be an important element of a utility company's service or of an industrial company's operations, a strictly non-condensing unit, or perhaps a certain small number of them, may be recommended, providing the electrical load reduces in the warm-weather months in a fair proportion to the loss of the heating demand of winter.

Criticism may be directed against the use of a large non-condensing steam turbine instead of a reciprocating engine. With recent advance in turbine design, it is difficult to show cause for criticism by stating that the engine consumes less steam when operating with atmospheric exhaust or higher back pressures. Any difference due to wear, may easily fall to the turbine's credit. Deducing from results obtained with a drum-type Parsons turbine, with 7 lbs. back pressure, the disparity is only about 5% at full rating, compared with an engine in excellent order. This difference vanishes on loads of less than one-half, or with poor adjustment and leakage by valves and pistons. A well-designed turbine should not perceptibly suffer in operation, and impartial tests confirm this fact. There is a saving in oil, labor and investment with the turbine, and a considerable reduction in maintenance expense of the exhaust mains, with freedom from oil, which will greatly overshadow the slightly better fuel economy of the engine.

Bleeder-type turbines owe their existence to the necessity for a mixed lighting and heating supply from a central power station. In some plants—chiefly large ones—the complete expansion turbines and the non-condensing type may be successfully co-ordinated to produce the highest economies in all directions. Moderate and small-sized stations, with a dissimilar fluctuation of lighting and heating loads throughout the day, month and year, would find it virtually impossible to regulate their equipment for constantly attaining the most efficient results. It would, moreover, probably call for a greater station investment to provide adequate flexibility. With compound condensing engines it has been a very general practice in cases of this kind to draw steam from the receiver, which becomes practical at all loads with engines having the cutoffs on the high and low-pressure cylinders parallel. In early turbine designs, an improvised expedient was followed to a partial extent by tapping steam from a given stage in which the pressure under any reasonable load would not fall below that maintained in the heating system, thus guarding against any interference with the supply or service. This method of operation was accompanied by two disadvantages; (1) a limited low pressure steam supply only being available through this means, and (2) a pressure reducing valve being necessary between the turbine and heating system.

Such diversified requirements in joint heat and electric service directed the attention of the turbine engineer to a very important problem, to devise a method by which both the heating and electrical supply would be automatically and economically delivered in accordance with the demands of the systems. The most effective and dependable solution has been the location of a pressure-controlled valve between the high and low-pressure sections of the turbine, automatically diverting to the heating system the exact amount of steam required and at precisely the predetermined pressure. This design is commercially designated the "automatic-bleeder turbine." Through the employment of a special

*From a paper before a convention of the Indiana Electric Light Association, Aug. 24, 1911.

†Westinghouse Machine Co., East Pittsburgh, Pa.

constant-pressure valve between a system of reciprocating engines and a low-pressure turbine, an exactly similar arrangement is secured, which deserves careful thought in the extension of the older plants containing steam engines.

DIVERSIFIED SERVICE.—Electric power production has created a demand for the turbines far in excess of all other stationary uses combined, and this results (1) since the turbine is pre-eminently a high-speed machine and its development has been accompanied by successful designs of high-speed direct-coupled generators, and (2) on account of the growth of electricity as a modern convenience. Lighting, being the greatest service, has brought the high frequency (60-cycle) units forward in moderate size stations. Large plants with a heavy direct-current load and rotary substations, mainly employ 25 cycles. Machines for 60 and 25-cycle service come within the realm of compatible speeds of the turbine and generator. Lower speeds, necessitated by 15-cycle and direct-current work compel a compromise of the efficiency and the mechanical structure of the two elements.

Large reduction gears have now become feasible, allowing the most desirable speeds of both turbine and generator simultaneously. However, direct-coupled generating units of about 300 KW. and under, are being suitably fitted to the needs of excitation sets and corresponding direct-current service where high economy is not essential.

Centrifugal boiler-feed pumps for boiler plants of above 2,000-HP. capacity, ranging in size from 15 to 100 B.HP., are being direct connected to the turbine. Coupling of the turbine to large direct-current generators, centrifugal pumps, screw propellers and other slow-speed apparatus is now successful with a large gear, and permits unrestricted latitude in design of the component machines. Use of a single reduction gear, or a train of gears has also brought the rolling mill requirements within the capabilities of the steam turbine.

MECHANICAL CONSTRUCTION.—In the reciprocating engine, the problems of design differ widely from those in turbine design. Reversal shocks and static forces, demand connecting rods, pins, crank discs, and such working parts, of very heavy construction in relation to the power transmitted. Turbines, on the other hand, convert the energy of the steam in many progressive steps, and at a high "material" speed, so that unit stresses in the various parts are comparatively low, notwithstanding the greater centrifugal forces. As an illustration, the hold and strength of the blade in the best Parsons designs, are forty times the forces tending to dislodge them in normal operation.

An obstacle in early turbine construction was the involved cylinder design which militated against uniformity in expansion and contraction of the parts. This was really productive of periodical blade trouble, the reason for which was very plain in the study the "old" line of reaction turbines. Early difficulties were due principally to the supports and equalizer ports being cast integral with the cylinder in all important sizes, producing variation in the depth of metal at different points, thus causing the cylinder to slightly camber with temperature changes. The trouble has been eliminated by separating these exterior parts from the cylinder proper.

There were cases of blade mishaps from contact due to the distortion above noted. Different qualities of blades have been employed until a successful composition and quality has been secured. Steel and copper-clad blades, which were used in certain stages of turbine history, soon gave out where the steam possessed chemical activity.

No mechanism is absolutely proof against all vicissitudes. Therefore some link in the turbine system should bear the brunt of any accident and should be restored with the greatest facility and at minimum cost. This should be the blading element; any attempt to unduly reinforce its construction may defeat the end in view. Obviously localizing all troubles in the blading would precipitate the least consequences—often too trifling to interrupt operation. The tremendous latent energy of high-speed disks, due to the depth of metal and its consequent weight, deserves careful thought in connection with the rupture or weakness of any section. No abnormal strains from temperature stresses should be introduced in the turbine in changing from condensing to non-condensing operation and vice versa.

Simplicity in valve and governor mechanism is essential for instant action at a critical moment. The governor or regulator should have forced lubrication and be encased for the safety of operatives.

When in service, the turbine should occasion a minimum of attention under any and all variations in load. It has scored over the reciprocating engine in the matter of small attendance, and this possibility of the turbine should not be neglected.

Power machinery is not for adornment, but exclusively for service. But effective outward design induces confidence in the installation and it frequently facilitates operation to remove ungainly parts from sight. All oil and water piping, cooling ducts and heavy valves should be imbedded below the floor line when their location above the bedplate serves no useful purpose. Such care in designs begets care in operation.

The design should permit of easy access to all the various parts so that it may be determined periodically whether the machine remains in normal condition or if small replacements or cleaning should be undertaken. When a large amount of dismantling is involved for mere inspection, the internal parts are liable to suffer neglect, either reducing the economy or causing the eventual repairs or damages to be excessive.

If the feed water is chemically active, it is important that the design freely allow lining the cylinder due to wall corrosion on account of the organic or inorganic elements present, whatever they may be.

RATINGS.—Within the last three years a new reference for rating generating units and other electrical power apparatus has come into use. This has taken the form of basing the full-load capacity on the greatest amount of power which may be delivered by the machine continuously without dangerous heating or strains or seriously falling off in speed. The capacity thus determined is termed a maximum rating. Previously, the more conservative practice provided all important machinery of this class with a continuous marginal overload of 25% and distinguished as the Normal Ratings. Each method of rating is to be respectively endorsed under appropriate circumstances. Only where there is definite knowledge that the unit will not be compelled to operate constantly at some greater capacity than fixed upon should maximum ratings be employed. This removes the conservative margin so essential in important service and should therefore be confined to special cases.

Turbines rated on a maximum basis are incapable of carrying full load should the vacuum be accidentally lost, which might embarrass the operation of the plant. The boilers should possess sufficient overload capacity to provide the increased steam required to run the turbine non-condensing. The boiler plant should not be rated at its maximum safe output, as a higher efficiency obtains at a lower rating.

The unit-capacity cost of a maximum rated turbine is necessarily lower than for a normal rated one with some overload capacity. This frequently leads to misapprehensions on the part of the purchasers as to the comparative value of new machines. An allowance for system power factors is also very important, its neglect in many plants having very unfortunately resulted in burning out the generator.

The proper measure of turbine performance is the ratio of the effective work to the heat energy actually absorbed. Water rates alone do not exhibit the true economy of the turbine, as low steam consumptions may be obtained by sacrificing the other station equipment. These facts are of more than technical interest and bear critical study. Moreover, they concern the operators as well as the designer and also prove helpful in gaging economy guarantees. Power engineers in their zeal to procure unreasonable efficiency often encourage hazardous guarantees. The latter practice has not been shunned as it justly deserves; conclusive tests are improbable in the majority of cases and there has not been sufficient attention to the fallacies of hazardous guarantees and erratic tests.

Many reliable records furnish fair standards of performance for different operating conditions. Therefore, it behooves those installing new turbines to specifically analyze the important features which underlie their plant. Penalty and bonus stipulations are only a mask unless the approved test conditions prevail and trials are conducted conscientiously and skillfully. Nevertheless, within reasonable limits, the award and penalizing on improvement or deficiency in guarantees on the whole is an excellent method of agreement and should be adopted and carried to conclusion in every possible case.

Metallised Paper is being manufactured by a German process, according to press reports, as a substitute for tinfoil. Ordinary paper is treated with a mixture of finely powdered metal and resin and then subjected to friction. The resulting products is claimed to be as flexible, brilliant and impermeable as tinfoil.

Unit Costs of Concrete Construction on the Panama Canal During the Fiscal Year, 1910-11.*

The total amount of concrete laid during the fiscal year 1910-1911 was 1,742,928 cu. yds. There were laid in the Atlantic Division 970,78[illegible] cu. yds., of which 59,651 cu. yds. were laid in spillway, and 911,137 cu. yds. in the locks.

In the Central Division, there was laid in connection with the flume constructed for the Obispo diversion 1,020 cu. yds..

In the Pacific Division, 771,120 cu. yds. were laid, of which 498,187 cu. yds. were placed in the Pedro Miguel Locks and 272,933 cu. yds. were placed in the Miraflores Locks.

In laying concrete for the locks in the Atlantic Division, the construction plant was operated on the basis of a 12-hour day, and the auxiliary plant on the basis of a 9-hour day, with the exception of a short period between September and November. In the Pacific Division, the work has been done on the basis of an 8-hour day. Taking into consideration the service time of hours at work, it is ascertained that the Atlantic Division laid 260,573 cu. yds. per hour, and the Pacific Division 281,299 cu. yds. per hour.

The cost per cubic yard for concrete in the Gatun Locks was $6.5919; in the Gatun Spillway, $6.7044; in the Pedro Miguel Locks, $4.7040 per cu. yd.; and in the Miraflores Locks, $4.6826. At Gatun, 73,609 cu. yds. of large rocks were used resulting in a saving of $263,137, or of 0.2888 cts. per cu. yd. on each yard of material placed.

In the production of stone for the concrete, the cost in bins at Gatun was $2.3403 per cu. yd., and in the storage pile for the locks on the Pacific side, $0.8443 per cu. yd. Sand was procured at Nombre de Dios for concrete in the Atlantic Division, and cost $1.8565 per cu. yd. in storage. Sand for the locks on the Pacific side was secured at Chamé and cost, in storage, $0.8284 per cu. yd. Crushed stone from Porto Bello is transported to Gatun in barges, and unloaded by cableways and derricks, while crushed rock from Ancon is transported from the quarry by rail to storage, and dumped from the trestles. The difference between the cost of towing and unloading, and that of transporting by rail, is $0.7184 per cu. yd. If this is deducted from the actual cost in storage, it leaves a cost of $1.6219 per cu. yd. for Porto Bello stone at Gatun, as against $0.8443 for stone for the locks on the Pacific side. The haul for sand from Nombre de Dios is 40 miles, while from Chamé it is 20 miles.

The Atlantic Division used cableways and cranes for unloading the sand, while the Pacific Division used electric cranes. Omitting the cost of transportation from the sand banks to the docks, the cost to the Atlantic Division was $1.3142 per cu. yd., and to the Pacific Division $0.6015 per cu. yd. Cement for the large part is delivered in barrels to the Atlantic Division at a cost of $1.19 at tide water in the United States, while in the Pacific Division, it is delivered in bags at a cost of $1.60 at tide water in the United States per bbl., less credit for return of bags. As approximately 90% of the bags were returned and accepted the cement in bags cost $1.01 per bbl. at tide water in the United States.

The construction plant in the Pacific Division handles a large percentage of its cement directly from cars to mixers resulting in reduced rates, whereas nearly all the cement in the Atlantic Division is handled through its storehouse. The year's operations show a difference in favor of the Pedro Miguel Locks of $1.7340 in the cost of cement, sand and stone, and large rock; in the other items which go to make up the cost of the finished product, the Pedro Miguel Locks show a lower cost for forms, placing, pumping, power, repairs, plant arbitrary, and in division expenses, while a difference exists in favor of the Atlantic Division in mixing and reinforcement. The construction plant at Pedro Miguel was in operation in its entirety from July 15 to Feb. 1, and a comparison of the costs for the six months' period, with the costs at Gatun for the year, show less costs at Pedro Miguel for all items, except for reinforcement.

In the item of mixing, the Pedro Miguel construction plant shows a cost of $0.1334, while at Gatun, the cost by the construction plant was $0.1749 per cu. yd.

Examination of the costs of the work shows also that the amount paid for salaries of clerks, and for supervisory forces, was 26.05% in the Atlantic Division, and 23.95% in the Pacific Division.

The Proposed New Charter for Atlanta, Ga., outlined on page 241 of our issue of Aug. 24, 1911, was defeated on Sept. 27 by a considerable majority.

*From the "Canal Record," Sept. 13, 1911.

The Partial Failure of a Concrete Dam at Austin, Pa., on Jan. 23, 1910.

(Reprinted from Engineering News of March 17, 1910.)

During the heavy floods which were prevalent throughout the eastern section of this country in the last part of January of this year, a number of sensational accounts appeared in press dispatches describing the failure of a concrete dam across Freeman's Run, in Austin, a town on the Buffalo & Susquehanna R. R., in Potter Co., Pennsylvania. These accounts consisted mainly in descriptions of the impending destruction of the town, which was saved only by the dynamiting of the dam holding back the flood. We have secured from Mr. T. Chalkley Hatton, M. Am. Soc. C. E., the designing engineer, an account of the trouble. From his notes we have prepared the following description.

The Bayless Pulp & Paper Co., of Binghamton, N. Y., operates a paper mill, securing water for its plant from Freeman's Run, which flows through the town of Austin. The paper mill and a large part of the town are located close to the banks of the creek, as the valley is very narrow and the surrounding hills steep and rugged, rising in a distance of one-half mile to a height of approximately 500 ft. above the level of the creek.

Freeman's Run has a watershed of approximately 35 sq. miles which is quite mountainous, and much of it is covered with a short growth of timber. During the summer and the fall the run-off from the stream is not sufficient to supply the paper mill with water for its purposes. About 11 years ago a small dam was built across this run about a mile above the mill to impound about 25 million gals. This dam was built with a rubble stone core wall, 5 ft. 6 ins. thick on the bottom and 18 ins. on top, with unpacked earth slope on the down-stream side built on a slope of 1:1 and an unpacked slope on the up-stream face of 1½ to 1. The core wall is 20 ft. high and rises 1 ft. above the normal water level in the dam, when the dam is full. It rests upon the top layer of sandstone rock which lies under the entire valley in thin layers.

To the eye this dam, when the water is out, looks very weak indeed, but it has successfully withstood the floods of 11 years, during which much of the down-stream slope has been washed away, exposing the core wall several feet in depth for most of its length, which is about 380 ft.

As the business of the paper mill increased, it became necessary to impound a greater volume of water in order to carry operations over low runs of the stream. Mr. Hatton was engaged to design and supervise the construction of the proposed dam, which he located about 800 ft. below the old dam where the width of the valley was about 350 ft.

To impound 200,000,000 gals. of water the dam was designed to carry water at a depth of 40 ft. above the normal level of the valley. During the construction, however, the owners directed that this depth be increased to 42 ft. to level of spillway with a 2-ft. 6-in. freeboard, and the accompanying figures show the cross-section as actually built.

The entire valley is underlaid with sandstone rock in horizontal layers running from 8 ins. to 3 ft. thick. Between these layers beds of shale and disintegrated sandstone were located, but there were very few vertical cracks to be observed in the bed of rock. Upon the surface of the rock there was from 5 to 8 ft. of earth and compacted gravel deposited from the washings of the side hills. This was so well cemented together that it was removed with the utmost difficulty, and but little water filtered through it into the foundation trench during construction.

In preparing the foundation, all loose rock was removed and the concrete only started after a solid stratum of at least 2 ft. thick was encountered. The surface of the rock was well washed and grouted. A cut-off wall 4 ft. thick and 4 ft. deep was built, as shown upon the plan (Fig. 1), and in every case this cut-off wall was carried down to good rock.

In order to reinforce the dam, twisted steel rods 1¼ ins. in diameter and 25 ft. long were built into the wall vertically 5 ft. inside of the up-stream face. Holes were drilled into the foundation rock from 5 to 8 ft. deep, and 2.66 ft. between centers and the steel rods with expansion bolt heads were placed into these holes, which were then grouted.

Upon each side of the spillway a pilaster was built which was reinforced by ½-in. steel rods to prevent cracking. The top 12 ft. of the dam was reinforced with ½-in. steel rods spaced 2 ft. vertically and 4 ft. horizontally to prevent cracking. Against the upstream face of the dam an earth embankment was laid at a slope of 3:1 and reached to within 27 ft. of the normal water level at the upstream face of the dam. This embankment was composed of disintegrated shale, clay and some loam, free from large stones, and was carefully compacted by grooved rollers in 6 to 8-in. layers, the work being carefully and well done.

The dam was composed of cyclopean concrete,

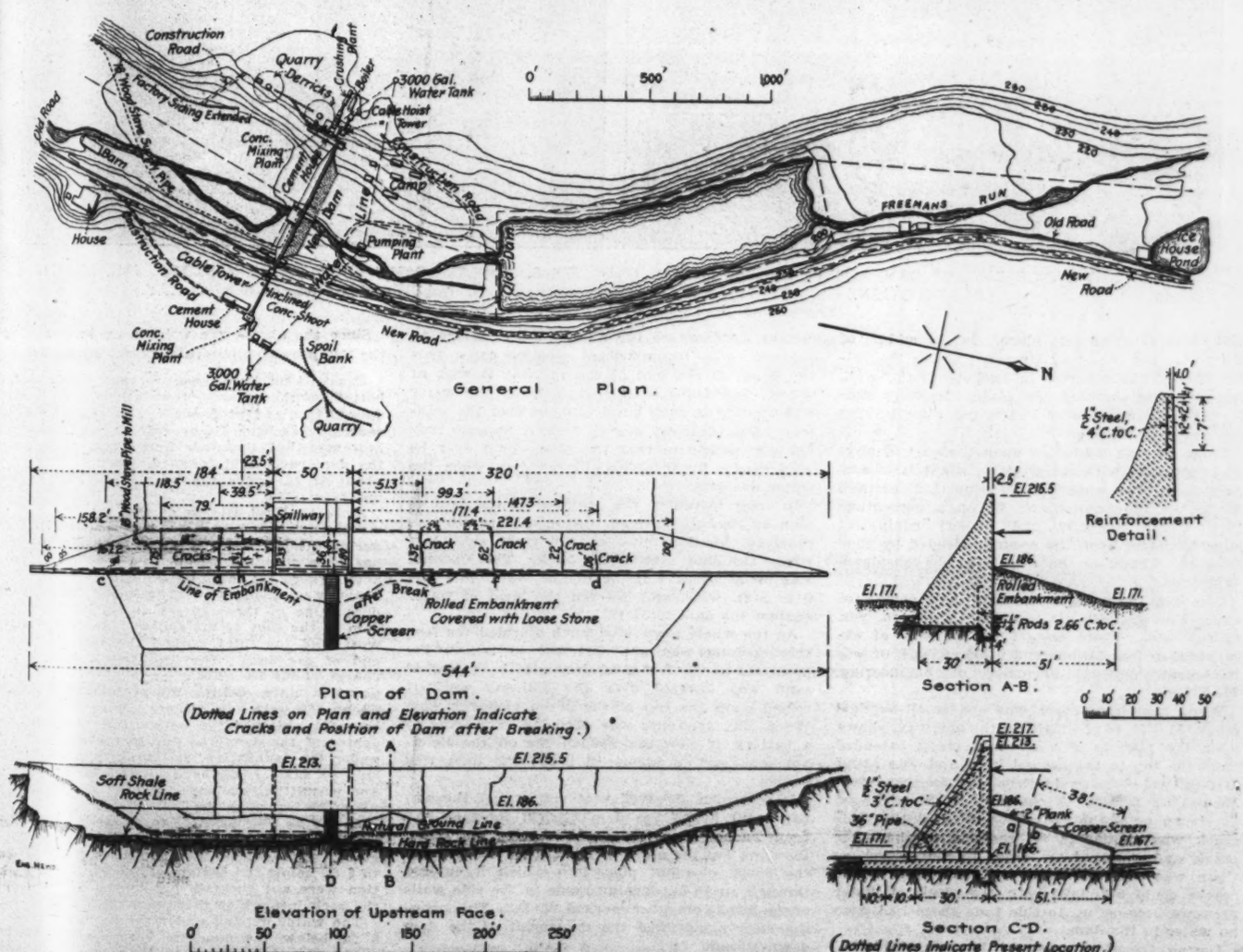

FIG. 1. GENERAL PLAN AND DETAILS OF CONCRETE DAM FOR THE BAYLESS PULP AND PAPER CO., AUSTIN, PA.
(Reproduced from Engineering News of March 17, 1910.)

with large quarry stones, from ½ to 2 cu. yd. size, embedded firmly in wet concrete. These stones were placed so as to break joints, both as headers and stretchers, and surrounded with not less than 6 ins. of concrete, generally much more. In every case the concrete was built in sections, stepped up as shown in Fig. 2, and the steps dove-tailed both horizontally and vertically. In building into the side hills great care was used in getting down to good firm rock foundation before any concrete was laid.

The material for building the dam was secured from quarries opened at either end of the dam, where duplicate crushers and mixing plants were located. The sand was secured by crushing the sandstone with rolls and screening through sand pans. The composition of the concrete was one part Portland cement, three parts of sand and six parts of broken stone. Each shipment of cement, as received, was tested upon the works by the resident engineer, who also made numerous tests to ascertain the difference between the tensile strength when using washed bar sand and the sand secured from crushing the stone. In every case the mixture containing the sand from the crushed stone gave the greater tensile strength.

The concrete was handled in two ways. At the two ends it was run down chutes upon the dam, filled to overflowing, which it was doing on Saturday, Jan. 21. On Sunday afternoon a large slice of the earth forming the eastern hill below the dam dropped down some 8 ft. and partially slid into the valley, and some water came down under the slide, which was evidently coming from behind the dam. Water in large quantities also began coming up through the ground from 15 to 50 ft. down-stream from the toe of the dam, showing that the water had gotten under the dam, through the embankment and through the rock strata.

On Monday a section of the dam between "a" and "b" (see plan view in Fig. 1) slid out at the bottom 18 ins. and at the top about 31 ins., this sliding being a maximum at a joint "h" and running out to nothing at "c" and "d." The crack at "a" opened 4 ins., at "h" 4½ ins. and at "e" and "f" 2½ ins. at the bottom on the down-stream face, but pinched nearly together on the upstream face. So far as the information was reliable, this movement covered a period of about 8 hours, when it stopped, and no further movement appeared, although the dam was overflowing after the movement had apparently ceased.

The 36-in. pipe which was built through the center of the dam as a draw-off had been capped by the company with a wooden cap attached to the bell of the cast-iron pipe with wrought-iron stream face," and these cracks were not over ½ to ¾-in. wide at the maximum, and usually much less. There was no indication of settlement except that the top of the dam at joint "h" had gone out 13 ins. farther than the bottom of the dam, which indicated some settlement of the toe of the dam at this point.

The dotted lines on Fig. 1 ("Plan of Dam") show the position of the dam when empty, as do the dotted lines in the cross-section marked "Section through C-D" (Fig. 1).

In the opinion of Mr. Hatton, the failure of the dam to withstand the flood of Jan. 21 is probably due to two causes. One of these is that the great bulk of concrete which had been hurriedly built, some during freezing weather, and which had been completed but six weeks before the maximum pressure came upon it, had not set up so as to attain its ultimate tensile strength. The most important cause, however, was due to the water getting under the dam. This condition was not anticipated when building the dam, and all precautions which seemed to be necessary at the time were taken to guard against it.

Nothing has been done toward reinforcing this dam. It is now being used, and at this writing there is a depth of 36 ft. of water behind it and no farther movement has been noticed.

FIG. 2. VIEW OF BAYLESS DAM, AUSTIN, PA., DURING CONSTRUCTION.

FIG 3. BAYLESS DAM UNDER MAXIMUM FLOOD, JAN. 28, 1910. (NOTE CRACKS BEYOND SPILLWAY.)

(Reproduced from Engineering News of Jan. 28, 1910.)

and there shoveled into place. In the middle of the valley the cableway stretched across the valley carried the concrete in buckets to the wall, where it was shoveled into place. In every case it was well turned over and mixed after leaving the mechanical mixer.

There was no indication upon the part of anyone connected with the work to slight it in any particular. The work was done by C. J. Britnall & Co., of Binghamton, N. Y., on a percentage plan. This company had a very competent superintendent upon the ground, assisted by good foremen. Therefore the work might be considered first-class.

The dam was begun in May and completed about Dec. 1, 1909. It contained 15,780 cu. yds. of concrete. There were 7,925 cu. yds. of excavation in foundations and 6,360 cu. yds. of embankment, costing, exclusive of engineering, $71,821.48.

When completed there was one small vertical crack, 51.3 ft. to the right of the spillway, shown upon the plan in Fig. 1. This crack extended from the top to the ground level and was about 1/16-in. wide. Later in December another crack showed up 39.5 ft. to the east of the spillway, as shown upon the cut above referred to. This crack was similar in all respects to the other crack, and appeared to be due to contraction, as there was no evidence of a settlement. These cracks could not have been the result of water pressure because up to this time there had been no water in the dam.

During the third week of January, 1910, the weather grew warm and some rain fell. The heavy snows of the preceding month were rapidly melting and within three days the dam clamps, contrary to the advice of the designing engineer, who recommended a valve cap. This cap being at the foot of the spillway it was, of course, impossible to remove it when the water was wasting in such large volumes over the spillway. The company had so built it because they did not anticipate that the pipe would ever be used except for cleaning out the dam when the water was low.

In order to relieve the pressure on the dam as soon as possible, a small section of concrete was removed, by dynamite, on the right side, just where the dam entered the bank. This opening was made about 4 ft. below the water line and 6 to 8 ft. wide, and lowered the head of water against the dam to 37 ft.

As the whole town was much alarmed for fear this opening was not sufficient to relieve the pressure, about 7 lbs. of dynamite in a bag of sand was lowered over the spillway until it rested upon the top of the 36-in. cleanout pipe. When this dynamite was exploded, by means of a battery, it blew the wooden cap off the cleanout pipe and in about 16 hours the dam was empty.

It was then observed that portions of the embankment above the dam against its face had been washed away, some of it going out under the dam, but the major portion going out through the 36-in. cleanout pipe, into which it entered through an 18-in. opening made in the side walls of the intake chamber marked "C-D." This opening clearly indicated the movement of the dam down-stream.

It was also observed that the upstream face of the dam was intact except for the vertical cracks marked on Fig. 1 "Elevation of Up-

Since the above was written we have received the following communication from Mr. Hatton:

Excavations have been recently made to the bottom of the concrete foundations upon the upstream face and no fracturing or crushing of the concrete has been discovered. It was discovered, however, that the rock foundation, upon which the dam rests, slid forward, starting at a point about 12 ft. above the base of the dam on the upstream side.

Since the first sliding of the dam no further movement of any character has occurred although during the past 15 days the water behind the dam has been standing within two feet of the overflow. There are three minor leaks under the dam showing up about 10 or 12 ft. below the toe of the dam. One of these shows as coming under the dam to the east of the spillway, the other to the west, and the third around the extreme eastern end of the dam. The first two appear to discharge about 300 gals. per min. and the other one as much more, making the present total leakage about 600 gals. per minute. From the evidence thus far obtained the outward movement of the center of the dam was due to the water getting under the foundation, softening up a stratum of clay or shale lying between two layers of rock, and permitting one layer of rock to slip forward about 18 ins. upon the lower layer.

So far as it is possible to determine the 1¼-in. steel vertical anchor bolts, which had been securely anchored to the foundation stone from 5 to 6 ft. below the bottom of the concrete foundation were not sheared, but moved outward with the rock into which they were anchored.

The water getting under the dam at a depth of 6 ft. below the concrete base exerted a much greater upward pressure than was anticipated when the cross-section was designed and reduced the factor of safety against sliding to an unsafe limit.

The Failure of a Concrete Dam at Austin, Pa., on Sept. 30, 1911.

On the afternoon of Saturday, Sept. 30, the worst disaster which has been recorded in this country in many years through the failure of an engineering structure occurred at Austin, Pa. The concrete dam of the Bayless Pulp and Paper Co. on Freeman's Run, just above the village of Austin, suddenly failed. The lake behind it, some 40 ft. deep at the dam and half a mile long, swept down the valley, destroying the larger part of the village of Austin (1⅛ miles below the dam) and much of the village of Costello (3 miles below Austin), causing a loss of life now estimated in the neighborhood of one hundred.

The cause of this disaster will be clear to every engineer from the account of a previous partial failure of this structure on Jan. 23, 1910, which was published in our issue of March 17, 1910. We reprint this article in full above, with its accompanying illustrations. We obtained this article in the following manner:

In February, 1910, local newspaper accounts reached us describing in language more sensational than technical the dynamiting of a portion of a concrete-dam to save it from destruction by flood.

This impressed us as a most unusual method of treating a dam, particularly one just completed, and we were especially curious to learn what sort of a dam it was which could be so imperiled by a flood. We therefore requested information from the designer of the dam, Mr. T. Chalkley Hatton, of Wilmington, Delaware, and received from him a statement with drawings and photographs from which the article referred to was prepared and published.

As related in that article, the dam was founded on strata of soft sandstone interspersed with layers of clay and shale and was carried down into the bed rock only about 4 ft. The dam itself was built very rapidly, part of it in freezing weather; but judging from the photographs of the broken dam which we publish in this issue, the concrete on the whole gave a very good account of itself, much better than rubble masonry dams have under similar circumstances.

The weak point was where one usually looks for trouble with masonry dams when trouble occurs—in the foundation.

We have heard intimations that Mr. Hatton, the engineer who designed the dam, desired in the original construction to carry a cut-off wall deep down through the soft rock strata so as to cut off possible percolation under the dam; but that he was overruled on account of the expense which would be involved. That he did fear percolation underneath the structure from the start is shown by the fact that he built an earth embankment against the upper face of the dam with as good material as was available in the vicinity hoping by this means to prevent water percolating down to the porous strata beneath the dam.

This means, however, proved futile. As stated by Mr. Hatton himself, as soon as the pond behind the dam filled, "water in large quantities began coming up through the ground from 15 to 50 ft. down-stream from the toe of the dam."

Within 24 hours after this flow under the dam appeared, the structure showed several great cracks and had moved down-stream, the maximum movement being about 18 ins. at the base and 11 ins. more at the top.

The people of Austin were greatly alarmed; and in the light of subsequent events they had reason to be. Apparently, the owners of the dam shared in this alarm. At least, they yielded enough to have a part of the dam dynamited near the top so as to somewhat lower the water back of the dam and reduce the pressure and also to blow off the cap of the draw-off pipe and empty the pond behind the dam entirely.

Shortly afterward they concluded (on what engineering advice, if any, we know not) that the dam was still strong enough to hold water safely. The draw-off pipe was closed and the reservoir was filled again. Within a month of the original failure they had raised the water within two feet of the overflow. This produced a leakage under the dam of about 600 gallons per minute; yet no one seems to have considered that of any importance as a danger warning.

FIG. A.— VIEW OF LOWER FACE OF DAM AFTER FAILURE, FROM EAST BANK.
(Photograph by the International News Service.)

That the owners had some ideas that they ought to do something to make their structure more respectable is shown by the fact that they did ask the advice of Mr. Hatton, the original designer, as to what repairs should be made. The facts concerning this are given in the following telegram from Mr. Hatton in response to our request for a statement from him:

> The account of the Austin Dam failure as published in issue of Engineering News of March, 1910, contains all the information I have up to the account of yesterday's terrible disaster except that Mr. E. Wegmann, Consulting Engineer of New York City, joined me in recommending method of reinforcing the dam. The plans and recommendation were submitted in February, 1910, since which date I have had no further connection with the dam and do not know what measures were taken to reinforce it.

The plan of strengthening proposed by Mr. Hatton was to sink a trench and build a cut-off wall above the dam to stop the water percolating beneath it and to reinforce the downstream face to prevent further movement. We reproduce here a cross-section made by Mr. Wegmann at the time, showing a proposal to reinforce the dam by dumping a heavy rock fill on the down-stream face as a means of satisfying the desire of the owners to keep the cost of repairs low.

It is to be especially noted that Mr. Wegmann was not asked to visit the dam site and view the condition on the spot; in fact, he was not employed by the owners of the dam at all, as we are informed, but by Mr. Hatton himself. He merely gave such general advice as was possible under such conditions.

It is a year and nine months since Mr. Hatton's recommendations for strengthening the structure were submitted, and the recommendations have never been carried out. The company's view of the matter is given in the statement of its president, Mr. G. C. Bayless of Binghamton, N. Y., as published in the New York "Times" of October 3.

Mr. Hatton's recommendations were made with the idea of making repairs to the dam so that the water could be increased to a depth of 50 ft., or so that if it ever became necessary to fill it up the

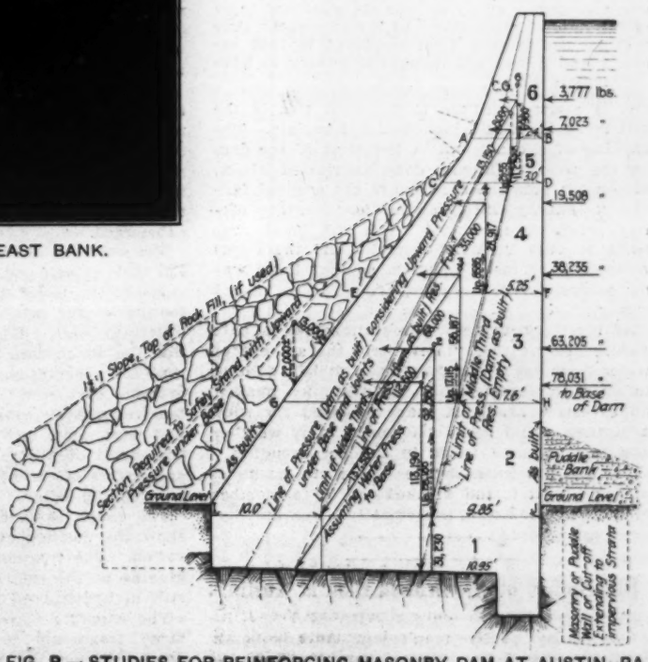

FIG. B.— STUDIES FOR REINFORCING MASONRY DAM AT AUSTIN, PA. FEB. 14, 1910: E. WEGMANN, CONSULTING ENGINEER.

	Explanatory:	
As built	Weight of masonry = 135.4 lbs. per cu. ft.	
	Total weight of section of dam 1 ft. thick	= 113,190 lbs.
	Less upward water-pressure	= 31,230 lbs.
	Net weight	= 81,960 lbs.
	Sliding resistance: 81,960 × 0.70 = 57,372 lbs.; or using 0.5 for wet foundation, 81,960 × 0.50	= 40,980 lbs.
	Resistance of rods = 35,000 lbs. per foot length (1¼-in. rods spaced 2 ft. 3 ins. c. to c.)	
	Total thrust, per foot length of dam, 78,031 lbs.	

Note: Upward pressure under base and joints of dam is assumed equal to two-thirds of hydraulic pressure due to head at up-stream face and decreasing uniformly to zero at down-stream face.

Fig. 1. View of Downstream Side.

Fig. 2. View of Upstream Side.

FIGS. 1 AND 2. FREEMAN'S RUN DAM AT AUSTIN, PA., AFTER FAILURE OF SEPT. 30, 1911.
(Photographs by Pictorial News Co.)

construction would stand the pressure. While we did not need 50 ft. of water we had thought well to provide for it in case of emergency.

This improvement was made by blasting out and removing a portion of the spillway, which was done in a manner to insure perfect safety. While certain repairs were made for the purpose we never attempted to carry 50 ft. of water in the inclosure.

We did not need that amount of water. Ordinarily we carried about 40 ft., which was sufficient to furnish us needed power.

We are going to detail expert engineers to the scene at once, who will make a scientific investigation. I believe that will be the best way in which to ascertain just what did cause the wall to break.

In the summer time, during dry seasons, we kept a man at the gate constantly to regulate the amount of water and keep it at a height that was necessary. This man was not stationed there at the time of the accident as it was past the time that he had usually been employed in that capacity and it was not thought necessary to have him remain there.

It is almost needless to say that this is the sort of explanation which does not explain. The blasting of a chunk out of the crest of the dam on the west bank was done as related above, during the panic at the time of the original failure in January, 1910. So far as "insuring perfect safety is concerned," this old break was partially built up with concrete, and the forms were in place for completion, so that there was no escape for the water through this alleged safety-valve.

As for the man who was stationed at a gate during dry weather "to regulate the amount of water;" whatever this may mean, it had nothing to do with safeguarding the structure against an excessive head. A man employed for such a purpose would be on the job in rainy weather and have nothing to do in summer droughts.

In the following article the facts gathered during a visit to the wrecked dam by a member of our editorial staff immediately after the disaster are recorded.

Description of the Broken Dam at Austin.

(By F. E. Schmitt, Editor Engineering News.)

The valley of Freeman's run extends south from the site of the wrecked concrete dam, 1½ miles to junction with a smaller valley of parallel course on the west. The main part of Austin occupies the valley bottoms at and just above the junction, but houses extend up Freeman's Run toward the Bayless pulp mill, half a mile above. The main valley has a flat bottom some 500 ft. wide for the entire distance between the dam and the village, the hills rising sharply on either side. Most of the houses are on the flat bottom ground, out of reach of any normal stream-flow but fully exposed to the flood wave caused by the failure of the dam. At and above the pulp mill, many thousand cords of pulp wood were stacked along the west bank. The flood wave carried away much of this wood, and thereby gained in destructive effect. The opinion locally is that the worst part of the destruction was wrought by the pulp wood.

The dam broke between 2 and 2.30 p. m., Saturday, Sept. 30, 1911. The appearance of the dam does not suggest a gradual failure; nevertheless, a telephone warning given from a house in view of the dam was circulated in Austin long enough to start many on the way to safety before the flood wave came. Most of the people of Austin had no warning. In the town the wreckage of the houses and logs from a kindling mill added further destructive elements to the wave. Extensive ruin was wrought three miles below in the village of Costello.

Examination of the broken dam on Oct. 2, 1911, shows that four-fifths of the length of 554 ft. is broken into seven large fragments, most of them still standing nearly vertical. The two largest are near mid-length, where they stand not far out of their original location, shifted 15 to 25 ft. downstream and swung only slightly from normal alinement. To either side of these is a large gap. The easterly gap is about 100 ft. wide, the westerly gap probably something over 50 ft. wide.

All the present flow of the stream goes through the easterly gap. The water here and in the pools along the dam is roughly at El. + 1.5 (zero datum being about at the bottom of the drain pipe, and crest at El. + 49). The standing fragments show the top of the footing course ½-ft. to 3 ft. above water, and their foundations are submerged.

The west end of the dam for a length of about 125 ft. is intact, and for 30 to 40 ft. farther east parts of the lower dam concrete or parts of the footing course are in place, at least along the upstream edge. East of this undisturbed section the entire dam is broken and displaced, except for a small piece, some 10 ft. long, at the extreme east end. The break-up being so extensive, and showing a large gap both at east and west ends, no offhand conclusion as to initial point of failure can be drawn. The whole appearance of the wreck, however, points directly to a sliding failure.

The rough sketches herewith (Figs. 5 and 6) show the position of the parts in plan and elevation. The fragments are lettered A to G, beginning at the east end; A and G are the parts still in undisturbed position.

The concrete appears sound, and each of the large fragments is a solid piece free from cracks, except for a group of broad cracks in fragment D, near the east end of the spillway; this group of cracks virtually divides fragment D into two pieces, connected only by a narrow band of concrete at the top. Seen from either side, the slope of the crest of D shows a sag over this group of cracks, though the two parts remain in alinement; and the easterly portion is a few inches lower than the other, as is seen near the bottom of the upstream face.

Only one of the larger fragments was evidently broken off elsewhere than at or near the bottom of the footing. This one is piece F', whose bottom corresponds to El. 15 to 20, approximately; three-fourths of this bottom appears to be the level surface of a horizontal construction-joint in the concrete. Piece F, adjoining E to the west, lies on its upstream face with crest heading north or upstream; its exposed bottom portion shows the footing split off on a very smooth horizontal mortar surface and this footing slab lies just below the corresponding base of F.

The other fragments did not permit conclusions as to their integrity down to foundation level, except in the case of C', whose overturned position exposes the bottom. A large flat slab of stone showing on this bottom face near the upstream edge, and the footing projection of the upstream face about 4 ft. up from the bottom surface, suggest that the actual foundation surface of C' is here visible.

Referring to the sketches Figs. 5 and 6, the order of the fragments in the original dam appears to be as lettered, i. e., A, B, C, C', D, E, F, and F', the last connecting to the undisturbed part G.

The east end of part G terminates with a fairly vertical break down to about El. 15, whence it shelves off in narrowing benches, the easterly ones showing as a projecting ridge in line of the upstream face of the dam. The highest shelf exposes the horizontally-bent (i. e., anchored) upper ends of the 1¼-in. twisted rods by which the dam was tied into the rock below. The lower shelves show the ends of a number of similar rods pulled out of the overlying concrete, and two or three others broken off with a well-contracted tensile fracture. The pulled-out ends retain part of the 90° bend by which they were supposed to be anchored in the dam. These rods are now bent over sharply downstream.

This shelving part of the remaining portion G extends across the gap between G and E, so that in this gap most of the separation surface is above the footing. Fragment F', probably the westerly piece out of this gap, obviously ends well above the footing, as above noted. Fragment F includes its footing at the toe as noted (the footing slab being split off, but lying directly beneath), but at the heel or upstream face it cannot extend so far down, as proven by the shelving remnants of the footing course just behind, which are still in true position.

Conditions at the easterly gap are radically different. Here fragment C is swung widely out of line, about 45°, and stands on its base with top slightly tilted upstream, but fully 4 ft. below correct level. Just to the southwest is the adjoining piece C', lying on its side, in a position which indicates that it shared the swing of C before falling over. The upper 15 ft. or more of C' is missing. East of C, piece B evidently shared in the swing until it reached firm bearing against the bank and permitted C to break off in a clean fracture. These pieces B, C and C' account for the gap east of D, and no important smaller sections seem to have broken out. There was no such shearing and splitting off as exhibited at the westerly gap.

The sketches Figs. 5 and 6 give some further information as to the position of the fragments, the water channels, etc., and show the sluiceway inlet in undisturbed position, separated by a diagonal break from the ends of its walls which remain attached to the upstream face of fragment D. The sketches show how much of the earth slope which existed along the heel of the dam is still in place. Along either side of the sluiceway inlet a narrow (10-ft.) gorge is cut

Fig. 3. View on Line of Original Upstream Face, From East End of Dam.
(Photograph by Pictorial News Co.)

through the earth bank, the sides nearly vertical, and the stream-flow now goes through these cuts. The easterly cut is crossed by a bar of rock or concrete about on line of the upstream face of the dam; the west cut carries most of the water.

of this basin as well as the surrounding wash have the appearance of being combed out radially by a violent outburst of water from the pool, occurring after the wash was deposited. At the east and west edges of this basin, against the concrete, remarkably upfolded and crumpled masses of laminated shale and stone strata appear, suggesting great downstream pressure. Similar crumpled rock strata show east of here, at the west edge of the channel, against D. The shale where found among the wash can be broken and crushed in the hand easily.

A number of the end faces of the main fragments are dark in color, and evidently represent the walls of old cracks. For instance, the east end of F', parts of the west end of E, the adjoining faces of E and D for their full height, and parts of the east end of D, show this discoloration. The east end of part G shows a clean fracture, and break B-C is fresh.

The sketches and photographs herewith should be compared with the sketch published in Engineering News, March 17, 1910, which showed the cracks existing in the dam at that time (reproduced on p. 417 of this issue). The west side of fragment E is easily identified as one of the old cracks by its peculiar shape, and the other of the old cracks are fairly well identified also by location or color, or both. The cracks shown in the "Elevation of Upstream Face" of our former article may be identified thus, passing from east to west:

(1)	?	
(2)	Break	A-B ?
	"	B-C New
(3)	"	C-C'
(4)	"	C'-D
(5)	"	D-E
(6)	"	E-F
(7)	"	F-F'
(8)	"	F'-G ?

The last is queried since the east face of the

Fig. 4. View on Line of Original Upstream Face, From West End of Dam.
(Photograph by Pictorial News Co.)

swung downstream and section B was held by jamming against the bank in front.

The position and character of the large cracks in section D have no such ready explanation.

Concerning the statements that repairs had been made after the slipping and cracking of the dam in January, 1910, no indications of repairs in the main part of the dam were observed. At the west end, where a short piece of the crest had been blasted out to afford an emergency overflow when the dam partially failed in January, 1910, there are in place wooden forms for bringing the concrete up to crest level, and some new concrete has been deposited, leaving the repair uncompleted. There are no indications that the cracks through the high portion of the dam were calked or otherwise improved. At the extreme east end, where the old drawing shows two cracks, there is now only the one break AB; similarly in the sluiceway walls, which show two cracks in the old drawing, no cracks are noted but a single straight break, not identical in location with the old cracks. Appearances point to some patching in the sluiceway walls, and possibly this explanation applies here and at the east end of the dam.

No indications of reinforcing, bonding or alterations of any kind to the original dam structure were observed. The dam was in the condition in which the accident of January, 1910, had left it, i. e., its main section was traversed by five or six cracks, and the central portion was (at that time) 1½ to 2 ft. downstream of its original location. In this condition it was recently subjected to a head of water nearly (or fully) as great as that which existed when the earlier slip occurred.

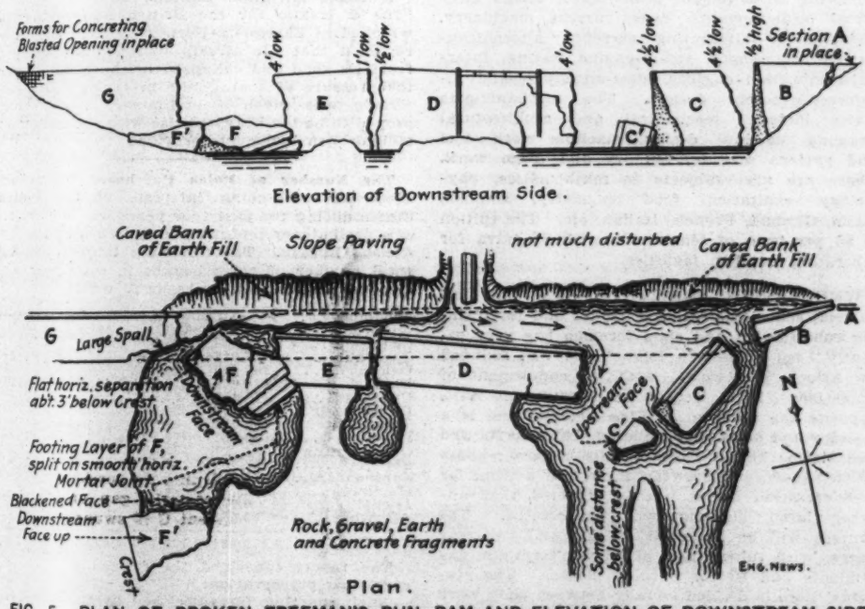

FIG. 5. PLAN OF BROKEN FREEMAN'S RUN DAM AND ELEVATION OF DOWNSTREAM SIDE.
(Drawn from rough sketches made on the ground Oct. 2, 1911.)

Besides the large fragments of the dam shown in the sketches, several smaller blocks lie one-hundred to several hundred feet downstream of the dam; these are not referred to on the sketches. A group of blocks lie 50 to 100 ft. south of fragment F', and others lie 100 to 200 ft. south of E. South of the pool downstream of F the sandy slope is very flat and suggests that a strong outrush occurred here, carrying the larger debris (including F') downstream or washing it toward the bank or toward the area protected by fragments D and E. This latter area, an acre or more, is covered to a depth of 3 to 5 ft. over the original sod with boulders, gravel, stone slabs, blocks of shale and sand.

Around the narrow gap between D and E, just downstream of the standing fragments, is an oval basin with steep sand slopes; the margins undisturbed part seems to be wholly fresh and clean, although the section between cracks (7) and (8) agrees roughly with piece F.

The principal new breaks, then, are those between fragments B and C, and that which partly divides section D near the east end of the spillway. The origin of the former may be explained (see above) as a transverse bending fracture which occurred when the combined section BC

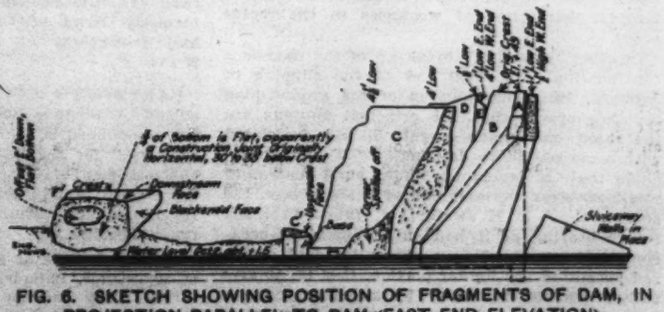

FIG. 6. SKETCH SHOWING POSITION OF FRAGMENTS OF DAM, IN PROJECTION PARALLEL TO DAM (EAST END ELEVATION).
(Elevations by hand-level comparison with crest of undisturbed end portions; offsets estimated.)

A timber crib dam now closes the valley about half a mile north of the concrete dam, with its base just above the latter's backwater. This seems to have been built within the last year or two. It may represent a means of providing water-supply when the pool behind the concrete dam should be drawn off for repairs, but it is certain that no repair work to the latter structure was going on at the time it failed.

The level of the pool at the time the dam broke was close to spillway level (El. 213, or 46½ ft. gage). Published newspaper accounts say, in effect, that water was running over the "dam" on the day of failure, and that this had not previously occurred since the accident of January, 1910. On Oct. 2, water marks on the east bank, when leveled against the undisturbed section A of the dam, showed 36 ins. below crest, which, assuming true level of crest, would be 6 ins. below spillway level.

The sketches, Figs. 5 and 6, give the height of crest of the several fragments below original crest level. The easterly end of B is the only one above level, all other portions being low by amounts ranging from ½ ft. to 4 ft. or over. The break D-E is a conspicuous high point; just in front of the dam at this place appears the oval basin already mentioned.

The elevation reproduced from our issue of March 17, 1910, gives no information as to levels of the different parts of the crest at that time. Except insofar as this is negative evidence, no means is at hand for judging as to any presumable lifting displacement at that period. The wreckage existing at present also gives no indication, unless the slight lift of the west end of fragment B be interpreted as a sign that possibly the original piece BC was lifted at its outshore end before or during its swinging movement.

Summarizing the indications of the wreckage of the Austin Dam, we may say that the westerly gap, with its sections split and sheared out at levels above the foundation, is clearly a secondary and subsequent effect. The origin of the failure lay between F and C. In the whole easterly section, all fragments (E, D, C', C, B) point downstream to a center at the east end of D. The gap between D and C now carries the stream-flow, and here also is presented the most extreme shifting of great masses. The break-up evidently had its most pronounced action here. But no localized failure of a single narrow portion would result in such extensive and coordinate movements as are evident.

It is recalled that, after January, 1910, water issued in considerable volume from the ground some feet downstream of the dam, at two points, to east and west of the spillway. This same flow doubtless continued, though unfortunately we have no later evidence concerning it; its washing effect would be continuous. The water exerting great hydrostatic pressure in the strata under the dam would reduce the frictional resistance at all times; how far along the dam such pressure effect might extend is only a question of circumstance. The effect of the conditions which existed 21 months ago was to produce maximum motion at the east end of the spillway, near where the new cracking now appears; and at that time obviously the foundation weakness was sufficiently distributed to permit sliding over the whole easterly two-thirds of the dam. There is no reason to expect a less extensive distribution of weakness in the recent event.

In every respect the break-up of the dam is a continuation to a final end of the slipping of January, 1910. Comparison of the ground-plan sketches proves that the principal motions are the same, only much magnified, and associated with some secondary destruction. We must conclude that, centering as before on a maximum weakness near the east end of the spillway, the section C C' D E swung out, failed to reach firmer bearing as it had on the previous occasion, and after sufficient separation had occurred permitted the lighter portion, B C C', to give fullest vent to the water by swinging far out. In spite of the old through crack between D and E these two parts moved together; the weakness of the underlying strata must have been very uniform indeed. That the jamming produced by the extensive motion split off two great fragments west of E will not appear surprising.

Soundings at and under the dam not having been made, it is not possible to judge how much of the underlying rock moved with the dam. Mr. Hatton in March, 1910, said that the rock foundation slid forward with the dam, the 5 to 6-ft. rod anchors taking the rock strata along in the slide. At present we lack a record of the depth and distribution of the rod anchoring, and of the shape of the heel cut-off extension and the contour of the footing level. The appearance of fragment C' suggests that at least at one point the cut-off did not afford a thrust bearing against sliding, and that here the dam did not take the underlying rock with it.

Full investigations of the disaster are promised by various authorities.

Notes From Engineering Schools.

COLUMBIA UNIVERSITY.—A course of lectures on City Planning, by Mr. George B. Ford, of New York City, is to be given at Columbia University during the first half of the year 1911-12. There will be no stated entrance requirement. The lectures will be illustrated with lantern slides, books and maps, and will be followed by a discussion, the lecture and the discussion to occupy two hours.

LEWIS INSTITUTE.—This school has issued a schedule of continuation courses to be given evenings and Saturdays for those who have no other opportunity to continue studies of value to them in their particular vocations. In previous years, some 1,800 students have been enrolled in such classes. The subjects in the engineering series include principles of design, electrical measurements, direct-current machinery, elements of alternating currents, alternating-current machinery, steam-engine testing, internal-combustion engines, steel-structure, and reinforced-concrete design. The mechanic-arts series includes mechanical and architectural drawing, machine design, machine work, tool and pattern making, foundry and forge work. There are also subjects in mathematics, physiology, sanitation, food chemistry, English, Latin, German, French, Italian, etc. The tuition is $5 per term of ten weeks, with $5 extra for laboratory or shop facilities.

WENTWORTH INSTITUTE.—This new industrial school for teaching trades and for training construction and shop foremen has very recently been opened in Boston. It was founded by Arioch Wentworth, with an endowment of $3,500,000. It is located on Huntington Ave., opposite the Museum of Fine Arts. There is a foundry and main shop building, 145 x 49 ft., and five stories high. There is also a power-house 80 ft. square and between these two a wing for demonstration offices. It is expected that another large building will be erected. The courses will be arranged on one and two-year bases, with tuition fees of $6 per term for day students and $3 for evening classes. The students' time is divided equally between shop work and class-room instruction. It is reported that some 600 applications for enrollment have been received. The Principal is Prof. A. L. Williston, formerly Head of the Department of Science and Technology at Pratt Institute, Brooklyn, N. Y.

DARTMOUTH COLLEGE.—The Amos Tuck School of Finance and Business Administration has announced a conference on "scientific management," for the benefit of manufacturers and business men of New Hampshire and neighboring states, to be held at Hanover, N. H., Oct. 12, 13 and 14. The following items appear in the program:

"The Principles of Scientific Management," Mr. F. W. Taylor, Philadelphia.

"The Task and the Proper Day's Work," Mr. H. L. Gantt, New York City.

"The Opportunity of Labor Under a System of Scientific Management," Mr. Harrington Emerson, New York City.

"Types of Management: Unsystematized, Systematized and Scientific," Mr. H. P. Kendall (The Plimpton Press), Norwood, Mass.

"The Spirit in Which Scientific Management Should be Approached," Mr. J. M. Dodge (Link-Belt Co.), Philadelphia.

"Round Table Conferences on Scientific Management in Machine, Textile, Shoe, Pulp and Paper Manufacture; in Printing, Publishing, Lumbering and Management of Timber Properties; and in Academic Efficiency."

The Application of the Scientific Method to State Activties," Mr. F. A. Cleveland (Bureau of Municipal Research), New York City.

"The Application of Business Methods to The Government of a Municipality," Mr. J. D. Burks (Bureau of Municipal Research), Philadelphia.

Another County-Fair Aviator. Cromwell Dixon, of New York City, employed by the Curtiss Exhibition Co., was killed as the result of a fall at Spokane, Wash., on Oct. 2. He had previously had trouble with his machine and the wind was coming in gusts at the time of capsizing.

An Automobile Bus fell from a bridge into the Seine River at Paris, France, Sept. 27. Of the 25 or 30 passengers in and on the bus, only ten were rescued alive, according to press reports, the others being imprisoned and drowned. The bus was crossing the bridge at high speed when the driver was forced to turn aside sharply to avoid a collision with another vehicle. The automobile skidded onto the sidewalk and broke through the railing at the edge of the bridge. The accident is ascribed by Paris newspapers to defective pavement and the sharp turn in the road approaching the bridge. A similar accident occurred at the same place in January. The bus was recovered from the river Sept. 28, and was found to be damaged only at the front end, none of the windows even having been broken.

A Rigid Dirigible Balloon, built by Vickers Sons & Maxim for the British Admiralty, was wrecked at Barrow-in-Furness on Sept. 24. It is reported that the airship was being towed out from its shed and collapsed in the middle under the pressure of a nine-mile breeze. In May, the airship was given a preliminary trial, but it proved to be too heavy to rise with the equipment of hydrogen balloonets provided.

The Number of Poles Purchased by various electrical wire-using interests of the United States during the past four years was the subject of a preliminary report recently issued by the U. S. Census Bureau. The following table shows the total number of poles bought in each year, and also the distribution by kinds of wood:

Kinds of wood.	Number of poles purchased. 1910.	1909.	1908.	1907.
Cedar	2,431,567	2,439,825	2,200,139	2,109,477
Chestnut	677,517	608,066	516,049	630,282
Oak	265,290	236,842	160,702	76,450
Pine	184,677	179,586	116,749	155,960
Cypress	75,459	77,677	90,579	100,368
Douglas fir..	56,732	24,877	19,542	15,919
Tamarack ..	30,964	29,889	24,123	13,884
Redwood	30,421	23,145	13,061	31,469
Osage Orange	23,221	21,491	18,109	5,962
Spruce	22,929	11,423	8,088	10,646
Juniper	20,042	43,581	42,367	38,925
Hemlock	12,773	6,222	1,998	3,301
Locust	9,030	10,463	10,224	4,672
All other.....	30,072	25,653	27,424	85,953
Total ...	3,870,694	3,738,740	3,249,154	3,283,263

The report contained the following statement regarding preservation:

An interesting fact disclosed by the figures is the rapid growth of the practice of treating poles in order to prolong their period of use. In 1910 824,673 poles, or more than 21% of the total reported purchases for that year, were given some preservative treatment. This number was an increase of 248,042 poles, or 43% over that of 1909; 480,285, or 139.5% over that of 1908; and 428,474, or 108.1% over that of 1907. In 1910 the class of pole users which applied preservatives most extensively was the electric railroad, light, and power companies, 29.4% of the reported purchases having been given some treatment of this character, as against 19.9% by steam railroads and 19.4% by telephone and telegraph companies.

The Pennsylvania R. R. opened a part of its Newark-New York City electric rapid-transit line on Oct. 1, and is now carrying passenger traffic from the Cortlandt St. (New York) terminal of the McAdoo tunnels to Manhattan Junction, where the extension for the 34th St. tunnels and ter-

minal leaves the old main line. Motor-car trains are operated over the lines of the Hudson and Manhattan R. R. (McAdoo tubes) to Jersey City and from there on the Pennsylvania R. R. right of way. There are at present 21 trains over this line into New York City and 11 out. The run is made in 16 min., compared with 21 minutes for the old schedule, which included change from a tube train to steam cars at the old Jersey City terminal.

A Noteworthy Bridge Alteration was carried out by the Atchison, Topeka & Santa Fe Ry. recently. The Red Rock Bridge over the Colorado River about 12 mi. east of Needles, Cal., has had its 660-ft. channel span divided into two 330-ft. spans by a center pier. The old span consisted of a 330-ft. suspended span carried on the ends of 165-ft. cantilever arms. The new pier supports the suspended span at its center, taking load when a train is on the bridge, with each of the new spans acting as a separate cantilever structure. By reinforcing various members, the bridge has been made strong enough to carry the new Santa Fe Mallet locomotives hauling 50-ton freight-cars. The pier foundations of the bridge are large enough to take a double-track structure, which may be built at a later period.

Personals.

Mr. W. L. Kellogg has resigned his position as Superintendent of Motive Power of the Cincinnati, Hamilton & Dayton Ry. to become Superintendent of Motive Power of the Pere Marquette R. R.

Mr. C. D. Young, M. Am. Soc. M. E., formerly Assistant Engineer of Motive Power of the Pennsylvania Lines West of Pittsburgh, has been made Engineer of Tests, with headquarters at Altoona, Pa.

Mr. Paul Hanson, formerly Chief Engineer of the Ohio State Board of Health, and recently State Sanitary Engineer of Kentucky, has been appointed Associate in sanitary engineering at the University of Illinois.

Mr. F. G. Barnes has resigned his position as Chief Estimator with James Stewart & Co., Contractors, of New York City, and has become Chief Engineer of the A. E. Stevens Co., with office at 25 East 26th St., New York City.

Mr. Warren B. Nissly has been appointed Superintendent of the Schuylkill Navigation Co., a subsidiary of the Philadelphia & Reading R. R., with offices at Philadelphia, Pa., to succeed Mr. Edwin F. Smith, who has been retired under the pension system.

Mr. J. P. Congdon, Assoc. M. Am. Soc. C. E., resigned on Sept. 30 his position as Supervising Engineer of the Oregon Short Line R. R. at Pocatello, Idaho. Mr. Congdon will enter upon a general engineering practice with offices at Boise and Pocatello, Idaho.

Mr. Charles L. Parsons, Secretary of the American Chemical Society, has been appointed Chief Chemist of the Mineral Division of the Bureau of Mines at Washington, D. C. Mr. Parsons has been until recently Professor of Chemistry in New Hampshire College at Durham, N. H.

Mr. Charles H. Miller, M. Am. Soc. C. E., formerly Engineer of River Protection of the Missouri Pacific Ry., at St. Louis, Mo., has organized the Miller Engineering Co., in partnership with Mr. A. C. Butterworth. The offices of the Miller Engineering Co. are in the Southern Trust Bldg., at Little Rock, Ark.

Mr. Alfred Craven, M. Am. Soc. C. E., has been appointed Chief Engineer of the Public Service Commission of New York City, with a salary of $15,000 per year. Mr. Craven had been Acting Chief Engineer, with a salary of $10,000, since the resignation of Mr. Henry B. Seaman, M. Am. Soc. C. E., in September, 1910.

Mr. S. K. Smith, M. Am. Soc. C. E., has resigned his position as General Manager of the Vinton Colliery Co., at Vintondale, Pa., and has opened an office in the Wright Bldg., St. Louis, Mo., as a consulting engineer. He will make a specialty of the examination, development and operation of coal properties.

Mr. A. M. Buck, Assoc. Am. Inst. E. E., who has been for the past year Professor of electrical engineering at the Clarkson School of Technology, at Potsdam, N. Y., has been appointed Assistant Professor of railway electrical engineering at the University of Illinois. He is a graduate of Cornell University, Class of 1904.

Mr. J. J. Corcoran, formerly Train Master of the Toledo division of the Cincinnati, Hamilton & Dayton Ry., has been appointed to succeed Mr. W. C. Hurst as Superintendent of that division. Mr. Hurst's retirement was noted in our issue of Sept. 14. Mr. Corcoran entered the employ of this railway in 1886 as a coach carpenter.

Mr. F. C. Lincoln, M. Am. Inst. M. E., has been appointed Associate in mining engineering at the University of Illinois. For the past year Mr. Lincoln has been in practice as a consulting engineer in New York City, and he was formerly Professor of geology of the Montana School of Mines. He is a graduate of the Massachusetts Institute of Technology, Class of 1900.

Mr. M. I. Forbes, Engineer of the Ashland division of the Chesapeake & Ohio Ry., has been made Engineer of the Cincinnati division to succeed Mr. F. H. Haskell, transferred. Mr. Forbes will be succeeded on the Ashland division by Mr. A. E. Botts, who was formerly Assistant Engineer of the Kentucky & Indiana general division. Mr. C. E. Rowland has been appointed to succeed Mr. Botts.

Mr. Francis W. Frost, for 18 years with the Engineering News Pub. Co., and for the last few years its Secretary and Treasurer, has resigned his position to enter the corporation of Suffern & Co., general importers and exporters, with offices at 96 Wall St., New York. Mr. Geo. H. Frost, formerly proprietor of Engineering News, and Mr. Edward L. Suffern, the well-known accountant, are directors of Suffern & Co.

Mr. L. D. Rowell, formerly Assistant Professor of electrical engineering at the Case School of Applied Science, has been engaged to take charge of the electrical laboratory of Purdue University at Lafayette, Ind. He is a graduate of the University of Wisconsin, Class of 1901. Mr. A. D. Du Bois will have charge of the junior electrical laboratory under Mr. Rowell's direction. Mr. Du Bois is a graduate of the University of Illinois, Class of 1899, and has recently been an instructor at Cornell University.

Mr. W. P. Hobson, formerly Master Mechanic of the Cincinnati division of the Chesapeake & Ohio Ry., has been appointed Superintendent of Motive Power of the Kentucky & Indiana general division to succeed Mr. W. T. Smith, retired. Mr. H. M. Brown has been appointed to succeed Mr. Hobson as Master Mechanic of the Cincinnati division. Mr. C. H. Doebler has been transferred from his position as Master Mechanic of the Huntington division. He has been succeeded at Chicago by Mr. H. C. Gillispie.

Mr. Richard C. Reinecke and Mr. Wm. A. Jenkinson, architects, of Sioux City, Iowa, have dissolved their partnership owing to the ill health of Mr. Reinecke, who has decided to retire. Mr. Jenkinson will continue the business in partnership with Mr. Milton J. Henoch, under the firm name of Jenkinson & Henoch, architects and engineers. Mr. Henoch is a graduate of Purdue University in civil engineering, class of 1898. Since Jan. 1, 1911, he has been engaged in grade-crossing elimination work in Memphis, Tenn., in the joint employ of the city and the several railways concerned.

Mr. M. S McDonald, formerly Superintendent of the Chicago division of the Chesapeake & Ohio Ry., has been appointed Superintendent of Stores, with headquarters at Huntington, W. Va. He will have charge of all stores of the railway company, both in the maintenance-of-way and motive power departments. Mr. J. A. Fox has been appointed to succeed Mr. McDonald as Superintendent of the Chicago division. Mr. Fox was formerly Superintendent of the Ashland division, where he will be succeeded by Mr. W. L. Booth. Mr. Booth's former position of Assistant Superintendent of the Cincinnati division has been abolished.

Prof. W. E. Adeney, F. I. C., Curator and ex-Examiner in Chemistry of the Royal University of Dublin, Ireland, will arrive in New York City on Oct. 5, to consult with the Metropolitan Sewerage Commission of New York on questions relating to the assimilation of sewage by harbor waters. Dr. Adeney is distinguished for experimental researches in chemistry, the results of which have been published largely in the transactions of the Royal Dublin Society and in the proceedings of the Royal Irish Academy. With Prof. Letts of Belfast, he contributed a notable monograph on the pollution of estuaries and tidal waters with a description of experimental researches into the phenomena of the digestion of sewage by water, which was published as Appendix VI. of the Fifth Report of the Royal Commission on Sewage Disposal. On the evening of Oct. 6, Prof. Adeney will be entertained by the New York Section of the American Chemical Society and will later make an address.

Mr. Wm. R. Copeland has resigned as Chemist in charge of the water-purification plant at Columbus, Ohio, and on Oct. 1 began service with the Metropolitan Sewerage Commission of New York (17 Battery Place, New York City), where he will have the title of Chemist. He will have charge of investigations which are being made by the Commission to throw light on the capacity of New York Harbor to assimilate sewage without offence. Mr. Copeland is a graduate of Harvard University and the Massachusetts Institute of Technology. From 1893-7 he was in general charge of the sewage testing station work at Lawrence, Mass.; during the next two years, he was in charge of the experimental water-filtration plant at Pittsburgh; from 1901-4 he was in charge of the biological work at the Spring Garden Testing Station of the Bureau of Water in Philadelphia; in the following year he was Bacteriologist at the sewage testing station at Columbus; from 1905-8 he was Assistant Engineer in the Bureau of Filtration in Pittsburgh; since 1908 he has been engaged at Columbus, as stated above.

Five new presidents have been named to head the various transportation systems known collectively as the Harriman Lines:

Mr. A. L. Mohler, Vice-President and General Manager of the Union Pacific Ry., has been made President of that company, and of the Oregon Short Line, with headquarters at Omaha.

Mr. Wm. Sproul, formerly President of Wells, Fargo & Co., has been made President of the Southern Pacific Ry., with headquarters at San Francisco. He has been succeeded as President of Wells, Fargo & Co. by Mr. B. D. Caldwell, Vice-President of the Delaware, Lackawanna & Western R. R.

Mr. Thornwell Fay, Vice-President and General Manager of the Southern Pacific lines in Texas and Louisiana, has been made President of those lines with headquarters at Houston and New Orleans.

Mr. F. C. Farrell, Vice-President and General Manager of the Oregon-Washington R. R. & Navigation Co., has been made President of that company, with headquarters at Portland, Ore.

Mr. Epes Randolph, Vice-President and General Manager of the Southern Pacific R. R. of Mexico, has been made President, with headquarters at Tucson, Ariz.

Mr. Robert S. Lovett, heretofore President of all the above-named lines, retains his position as chairman of the executive committee of the Union Pacific Ry., with chief executive authority over the whole system.

Obituary.

Rear-Admiral Winfield Scott Schley, U. S. N., retired, who was in command of the victorious American fleet at Santiago on July 3, 1898, dropped dead on the street in New York City Oct. 2. Rear-Admiral Schley was born in Frederick County, Maryland, in 1839.

Alonzo P. Blakslee, who was a division superintendent of the Lehigh Valley R. R., for twelve years, died Oct. 2. Mr. Blakslee was born at Mauch Chunk, Pa., on March 13, 1847, and he entered railway service in June, 1862, in the construction department of the Lehigh Valley R. R. He became train dispatcher of the Mahanoy Division in 1869, and was promoted to Assistant Division Superintendent in 1879. In 1890 he was made Superintendent of the Pottsville Division of the same railroad. From February, 1892, to April, 1898, he was Superintendent of the Mahanoy, Beaver Meadow and Hazelton divisions. In 1905 he was made General Manager of the Mauch Chunk, Summit Hill & Switchback R. R.

Engineering Societies.

COMING MEETINGS.

AMERICAN ELECTRIC RAILWAY ASSOCIATION.
Oct. 9-13. Annual convention at Atlantic City, N. J. Secy., H. C. Donecker, 29 West 39th St., New York City.

NATIONAL ASSOCIATION OF RAILWAY COMMISSIONERS.
Oct. 10. Annual convention at Washington, D. C. Secy., Wm. H. Connolly, Washington, D. C.

AMERICAN INSTITUTE OF MINING ENGINEERS.
Oct. 10-17. Annual convention at San Francisco, Cal., followed by trip to Japan. Secy., Joseph Struthers, 29 West 39th St., New York City.

RAILWAY SIGNAL ASSOCIATION.
Oct. 10-12. Annual convention at Colorado Springs, Colo. Secy., C. C. Rosenberg, Bethlehem, Pa.

AMERICAN RAILWAY BRIDGE AND BUILDING ASSOCIATION.
Oct. 17-19. Annual convention at St. Louis, Mo. Secy., C. A. Lichty, C. & N. W. Ry., Chicago, Ill.

AMERICAN GAS INSTITUTE.
Oct. 18-20. Annual convention at St. Louis, Mo. Secy., A. B. Beadle, 29 West 39th St., New York City.

AMERICAN MINING CONGRESS.
Oct. 24-28. Annual session at Chicago, Ill. Secy., J. F. Callbreath, Denver, Colo.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.
Nov. 2-4. Annual meeting at Cincinnati, Ohio. Secy., Edward H. Reisner, 20 West 44th St., New York City.

NATIONAL MUNICIPAL LEAGUE.
Nov. 13-16. Annual meeting at Richmond, Va. Secy., Clinton R. Woodruff, 705 North American Bldg., Philadelphia, Pa.

AMERICAN ROADBUILDERS' ASSOCIATION.
Nov. 14-17. Annual convention at Rochester, N. Y. Secy., E. L. Powers, 150 Nassau St., New York City.

AMERICAN RAILWAY ASSOCIATION.
Nov. 15. Semi-annual meeting at Chicago, Ill. Secy., W. F. Allen, 75 Church St., New York City.

NATIONAL FOUNDERS' ASSOCIATION.
Nov. 15-16. Annual convention at New York City. Secy., F. W. Hutchings, 915 Hammond Bldg., Detroit, Mich.

SOCIETY OF NAVAL ARCHITECTS AND MARINE ENGINEERS.
Nov. 16-17. Annual meeting at New York City. Acting Secy., D. H. Cox, 29 West 39th St., New York City.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
Nov. 17-18. Annual meeting at Canton, Ohio. Secy., F. E. Sanborn, Ohio State University, Columbus, Ohio.

NEW JERSEY SANITARY ASSOCIATION.
Nov. 24-25. Annual meeting at Lakewood, N. J. Secy., J. A. Exton, 75 Beech St., Arlington, N. J.

AMERICAN PUBLIC HEALTH ASSOCIATION.
Dec. 4-9. Annual meeting at Havana, Cuba. Secy., Wm. C. Woodward, Washington, D. C.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
Dec. 5-8. Annual convention at New York City. Secy., Calvin W. Rice, 29 West 39th St., New York City.

American Society of Engineer Draftsmen.

The first annual meeting was held at Teachers' College, Columbia University, New York City, Oct. 3. The Secretary is Mr. Walter M. Smyth, 116 Nassau St., New York City.

Western Society of Engineers.

At a special electrical meeting, held Sept. 27 in Chicago, Ill., Mr. S. G. Neiler presented a paper on "The Electrical and Mechanical Equipment of the New Terminal, C. & N. W. Ry."

American Society of Civil Engineers.

At the regular meeting, held in New York City on Oct. 4, a paper by Mr. Wilson Sherman Kinnear, entitled "The Detroit River Tunnel," was presented for discussion. This paper was printed in the August "Proceedings."

Albany Society of Civil Engineers.

The regular September outing was held Sept. 30. The party of members and guests inspected the new shops of the Delaware & Hudson R. R. now under construction at Watervliet, N. Y. Upon returning from Watervliet the party visited the municipal filtration plant at Albany. A special demonstration of the operation of the plant was made under the direction of Mr. H. J. Deutschbein, Superintendent of the Bureau of Water.

American Society of Engineering Contractors.

At the regular monthly meeting in New York City, on Sept. 12, Mr. Myron H. Lewis gave an illustrated lecture on "Waterproofing as a Means of Insurance." Mr. Lewis contended that the use of special waterproofing compounds seldom accomplished the desired result of making the concrete water-tight, although it might be true that they would prove successful were the concrete properly graded, mixed and laid. Even assuming, he said, that concrete is dense enough to prevent the passage of water in any appreciable quantity, capillary attraction makes it impossible to prevent the presence of moisture which is in itself highly objectionable. The principal object of waterproofing compounds should be to eliminate the porosity of concrete so that it would not absorb water.

Association of Iron and Steel Electrical Engineers.

The fifth annual meeting of the Association of Iron and Steel Electrical Engineers was held at the Hotel Imperial, New York City, on Sept. 25-30. In addition to the regular business sessions and inspection excursions to points of engineering interest, the following program was carried out:

Report of Safety Committee and Discussion. Description of Safety Devices Used in the Iron and Steel Industry, F. W. Woodhull (Luken Iron & Steel Co.); "Dynamic Safety Stops for Electric Cranes," Cutler Hammer Mfg. Co.; "Recent Developments in Mill Control," M. A. Whiting (General Electric Co.); "Automatic Control of D. C. Motors," A. C. Eastwood (Electric Controller & Mfg. Co.); "Direct-Control Reversing-Motor Drive for Planers," G. W. Richardson (American Bridge Co.); "Transformers for Steel-Mill Service," H. C. Soule (Westinghouse Electric & Mfg. Co.); "Illustrated Lecture on Safety," R. J. Young (Illinois Steel Co.); "Electric Cranes for Steel-Mill Service," E. Friedlaender (Carnegie Steel Co.); "Methods of Lubrication for Electric Cranes," T. E. Tynes (Lackawanna Steel Co.); "Track Wheels for Electric Cranes," W. T. Snyder (National Tube Co.); "A. C. Versus D. C. Cranes," K. A. Pauly (General Electric Co.); "Recent Developments in Motors for Steel-Mill Service," B. Wiley (Westinghouse Electric & Mfg. Co.); "A. C. Versus D. C. Motors for Auxiliaries of Steel-Mill Apparatus," B. R. Shover (Carnegie Steel Co.) and E. J. Cheney (General Electric Co.); "Design of an Electrically Driven Central Pumping Station," F. W. McKee (Jones & Laughlin Steel Co.); "The Regulating Storage Battery in Steel-Works Service," F. W. Woodhull (Lukens Iron & Steel Co.); "Prime Movers in Central Power Stations," Westinghouse Machine Co.; "The Light for Safety," Mr. Fortune (Cooper-Hewitt Electric Co.); "Illumination of Steel Works," H. M. Gassman (Tennessee Coal, Iron & R. R. Co.); "Steel Works Illumination," C. J. Mundo (General Electric Co.); "Some Features of Good Steel Mill Illumination," Ward Harrison (National Electric Lamp Assoc.).

The majority of the papers presented were descriptions of apparatus devised by various electrical manufacturers and steel-works engineers for service in various departments of a steel mill. A few of the papers aroused considerable discussion, for instance: the paper by Messrs. Shover and Cheney on "Alternating-Current vs. Direct-Current Motors for Steel-Mill Auxiliaries." The authors of this paper tabulated the first costs and operating expenses of a system involving the use of alternating current only and of a system with alternating current for transmission and direct current for the motor drives, rotary convertors being interposed between the generators and the motors. The figures of the authors in general showed that the straight alternating-current system cost slightly more than the mixed system to install, but that its operating and maintenance costs were lower than with the mixed system, while there was a gain in production through the elimination of certain maintenance delays. A number of the other engineers present did not agree with these conclusions and pointed out certain figures that had not been given which were to be included in the expenses of the straight alternating-current system, and held that the claimed freedom from interruption with the straight alternating-current system was more fancied than real.

The number of papers on mill illumination and the discussion following them indicated that the members of the association were alive to the advantages which accompanied adequate lighting and to the need of maintaining fixtures in good condition and of keeping the mill buildings clean and clear to promote efficient and effective lighting.

American Society of Municipal Improvements.

The eighteenth annual convention of the society was held at Grand Rapids, Mich., Sept. 26-29. A considerable part of the time of the convention was devoted to roads and pavements. In fact, on the day before the convention opened 50 men interested in paving specifications participated in open committee meetings dealing with the subject. These committees reported during the convention.

With the exception of the Secretary, Mr. A. P. Folwell, and the Treasurer, Mr. E. S. Rankin, Newark, N. J., who were retained in office, the officers of the past year were advanced one step up the presidential ladder, Mr. Fred Giddings, of Atchison, Kan., being thus deposed from the top rung and being succeeded by Mr. E. A. Kingsley, of Little Rock, Ark. Mr. Maury Nicholson, City Engineer of Birmingham, Ala., was elected Chairman of the Finance Committee.

Standard specifications for brick and for asphalt pavements, differing but slightly from those of the Association for Standardizing Paving Specifications, were adopted. The society also adopted the standard methods of testing asphalt already approved by a sub-committee of the American Society of Testing Materials.

The society made available some $750 to aid in devising standard methods and forms for municipal accounting and for the recording and comparison of physical statistics.

On recommendation of Mr. James C. Hallock, Deputy Chief Engineer, Newark, N. J., who is Chairman of the society's Committee on Municipal Data and Statistics, the society adopted the schedule for a uniform summary of water-works statistics, formulated many years ago by the New England Water Works Association, endorsed, with slight modification, a few years ago by the American Water Works Association.

The next convention will be held at Dallas, Texas, in November, 1912, the exact date being left open.

One of the most interesting of the papers on pavements was presented by Mr. H. [illegible] Lykken, of Grand Forks, N. D. This paper was entitled "Bituminous Concrete Pavements." Although less militant in character and taking up some other phases of the subject, it was similar in spirit to a paper presented before the same society at Erie, Pa., a year ago, by Mr. E. A. Kingsley, of Little Rock, Ark. Like the paper by Mr. Kingsley, the one presented this year by Mr. Lykken brought Mr. Geo. C. Warren to the front in defense of the so-called bitulithic patents controlled by the company with which he is connected. Mr. James S. Head, attorney for the same company, reviewed court decisions in support of the patents.

Mr. Lykken maintained that there was a natural transition from cement-mortar to concrete by the addition of broken stone, and that in much the same way sand-asphalt paving surfaces had been naturally transformed into bituminous concrete by the addition of stone. He remarked that no one thought of patenting the graded mixture of the concrete, and left his hearers to infer that there was no more reason for taking out a patent on the grading of the aggregate in bituminous concrete. He stated that "the evolution of the coarser mixture from the finer was analogous and inevitable in either case." Having set forth his views on this part of his subject, the author discussed various practical phases of bituminous concrete, including the choice of the bituminous material, which, in his opinion, should be carefully selected asphalt. In short, a bituminous pavement, from Mr. Lykken's viewpoint, would be composed of an asphalt-mortar with crushed stone added, the size of the stone to be approximately one-third of the depth of the pavement layer.

In a lengthy written discussion of Mr. Lykken's paper Mr. Warren contended that there should be a clear-cut distinction between mortar and concrete. He maintained that no such analogy exists between hydraulic cement-concrete and bituminous cement-concrete as had been claimed by Mr. Lykken, and he also maintained, as one would naturally expect from the representative of a proprietary process, that the addition of stone to bituminous mortar, instead of being merely an obvious step in the evolution of pavements, was a highly useful and meritorious patentable invention. He also urged that the grading of the stone in bituminous concrete has an altogether different function from similar grading in Portland cement concrete.

Another paper in the Warren interests was presented by Mr. August Eduard Schutte. This paper was entitled "Development of Bitucrete (Double Bond) Surface." Starting with a concrete foundation while the latter is still plastic, the wearing surface under description is prepared by the application of a layer of uniform-sized stone, say ½ to 1-in. in diameter, which has been previously coated with a Portland cement grout, either of neat cement or else of cement with a slight amount of sand. It is possible to coat this stone in place. After this layer has been down two or three days, a tough bituminous compound is poured upon it, sufficient to fill the voids and leave a layer spread over the surface of the cement-coated stone. Upon this bituminous surface ¼-in. stones are spread, apparently so rolled as to key into the stone already placed. Finally, sand or stone screenings are spread over the whole in order to make a non-slippery surface. By the means described, the author claimed that there is formed a "waterproof, non-absorbent, semi-elastic surface, which can be made as smooth or as rough as the conditions, or rather the engineer, may require." No special machinery is needed; first cost is low and repairs are easily made. "Bitucrete" was recently put on the market after having been tested at Cambridge, Mass. It has been put down in Newton, Mass., and in Morristown, N. J.

A list of the papers presented at the convention, as then scheduled, appeared on p. 326 of our issue of Sept. 14, 1911. Among several additional papers was one on "Physical Records of Municipalities," by Mr. E. C. Meyer, of the U. S. Bureau of the Census, Washington, D. C., and "Standard Tests for Asphalt Cements for Sheet Asphalt Pavements," by J. W. Howard, New York City.